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RESEARCH ON THE CONSTRUCTION OF EMBANKMENTS

A REPORT ON COOPERATIVE RESEARCH IN INDIANA AND OHIO

By HENRY AARON, Highway Engineer, Division of Tests, Public Roads Administration, W. T. SPENCER, Soils Engineer, State Highway Commission of Indiana, H. E. MARSHALL, Geologist, Ohio Department of Highways

FIGURE 1.—EMBANKMENT SITE IN INDIANA (A) AFTER CLEARING AND (B) AFTER HEAVY RAINS WHICH OCCURRED DURING JULY 1938.

CONTROL OF EMBANKMENT CONSTRUCTION in accordance with laboratory compaction test data is a relatively recent development. The methods of performing both the laboratory tests and the field control tests vary with different organizations,[1] but in all cases the test results disclose the relation between moisture content and density of a soil which results from a given method of compaction. A standard procedure for making the test for compaction and density of soil was adopted by the American Association of State Highway Officials in 1938[2] and by the American Society for Testing Materials in 1942.[3]

This test discloses the moisture content of a soil at which the density obtained by a given method of compaction is higher than for any other moisture content. This is called the optimum moisture content. Greater stability of fills has been obtained wherever this test has been adopted as a basis for controlling compaction. However, many questions of importance arose in connection with its practical application.

Among these were the efficiency of different types of rollers in obtaining the desired results, the relation of the depth of layer to type of roller, the economics and limitations of moisture control, and the performance of the completed fill with respect to settlement and change in stability. In an effort to find answers to these questions, two cooperative projects, one by the State Highway Commission of Indiana and another by the Ohio Department of Highways, were undertaken with the Public Roads Administration cooperating.

As a result of this work, two independent reports were prepared entitled Fill Construction Experiment in Indiana and Fill Construction Experiment in Ohio. W. T. Spencer, soils engineer of the State Highway Commission, participated in the preparation of the Indiana report. H. E. Marshall, geologist of the Department of Highways, participated in the preparation of the Ohio report. Lt. Col. R. R. Litchiser, former chief engineer of tests, and K. B. Woods and Capt. C. H. Shepard, former assistant engineers of the Ohio Department of Highways, co-operated in planning and directing the Ohio project. For the Public Roads Administration Lt. Henry Aaron, former highway engineer, participated in the preparation of both reports.

The experiments were similar in a number of respects including principal objectives, soil variables, control

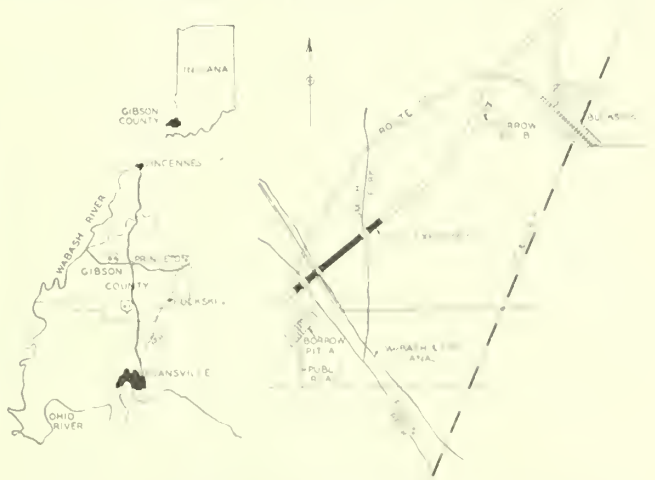

FIGURE 2.—LOCATION OF FILL CONSTRUCTION EXPERIMENT IN GIBSON COUNTY, INDIANA.

[1] Proceedings of the Eighteenth Annual Meeting of the Highway Research Board. Compaction of Earth Embankments, Part II (1938).

[2] Standard Specifications for Highway Materials and Methods of Sampling and Testing, published by the American Association of State Highway Officials, 1220 National Press Building, Washington, D. C., 1938 and 1942 Editions, Method T 99-38.

[3] A. S. T. M. Standards Designation D 698-42 T, Tentative Method of Test for Moisture-Density Relations of Soils, published by the American Society for Testing Materials, 260 South Broad Street, Philadelphia, Pa. 1942.

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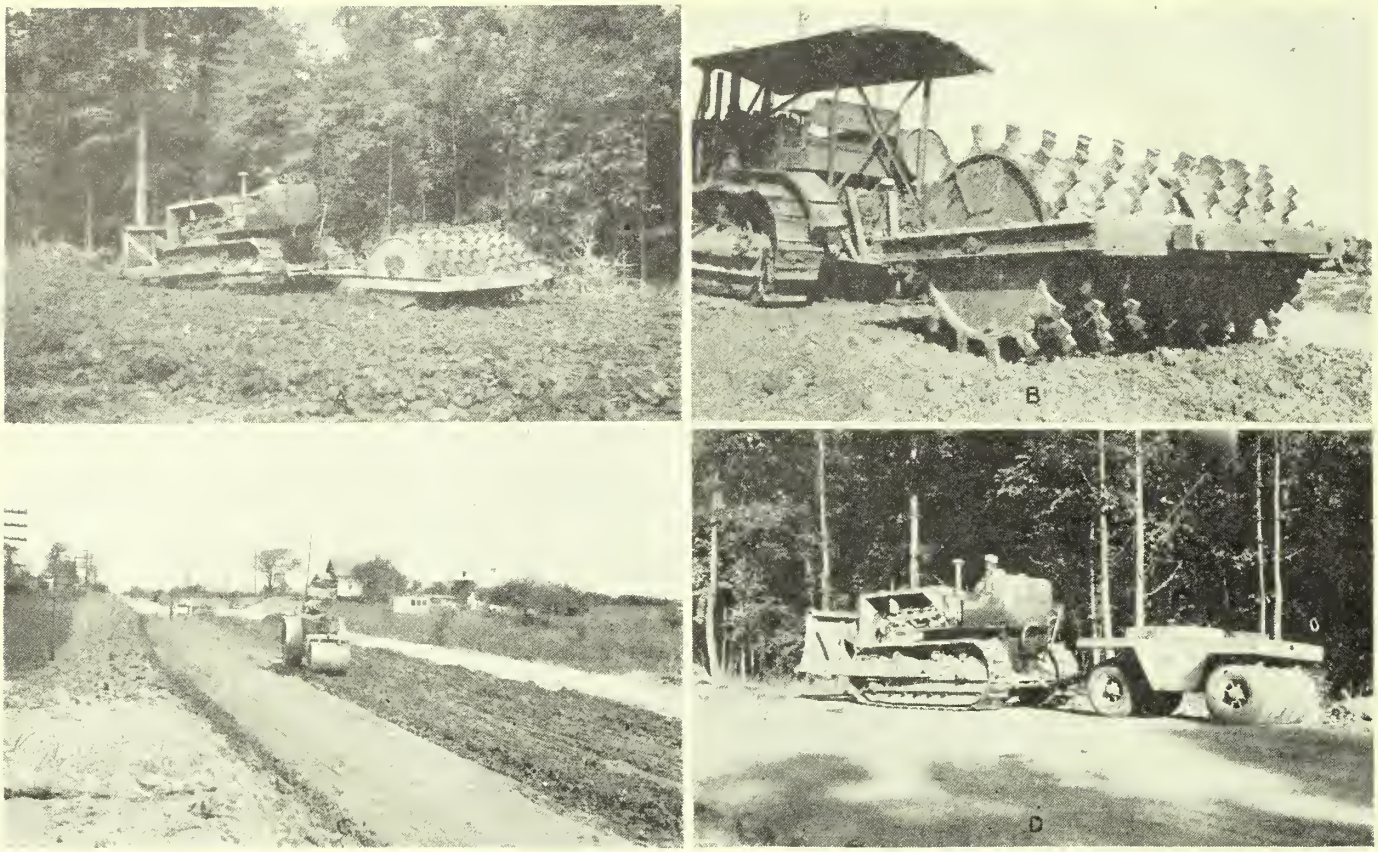

FIGURE 5.—TYPES OF ROLLERS USED (A) SHEEPSFOOT ROLLER TYPE A IN INDIANA, (B) SHEEPSFOOT ROLLER TYPE B IN OHIO, (C) THREE-WHEEL, 10-TON ROLLER IN OHIO, AND (D) PNEUMATIC TIRE ROLLER IN INDIANA.

pounds per square inch. The roller had a net weight of 2,680 pounds, but the platform was loaded so that under working conditions the roller gave a pressure of about 225 pounds per inch of width of tire surface in contact with the ground

TESTS OF SOILS SHOW A CONSIDERABLE RANGE IN PROPERTIES

Prior to the construction of the embankment, subgrade surveys were made on both projects to determine the character of the soil in the fill areas and in deposits available for excavation.

Indiana Experiment.—Borings were made with soil augers and representative samples, obtained from the different soil layers, were tested in the laboratory of the State Highway Commission. The results of these tests are given in table 4.

The soil profile along the center line of the project is shown in figure 3. The top 3 to 4 feet of the soil in the embankment area varies from a gray friable silty clay loam to silty clay. This is underlaid by a more compact, mottled material of the same texture. At depths of from 9 to 12 feet, it grades into a compact, impervious silty clay. Free water was found in many places on top of the compact layers of soil. According to the test data shown in table 4, the soils have physical properties of the A–4 and A–7 groups.

The material used in the embankment was obtained from three sources in the upland areas adjacent to the fill locations; the cut from station 238 to station 241, borrow pit A about 1,000 feet right of station 246, and borrow pit B to the right of station 336. The borrow pit locations are shown on figure 2.

Soil profiles at the three sources were similar in character. The upper layer, about 12 inches thick, was a brownish friable silt loam or silty clay loam having physical properties of the A–4 group. The underlying soils to depths of more than 14 feet varied from a mottled gray and brown to a grayish silty clay or clay, which was friable when dry, crumbly when moist, plastic when wet, and had physical characteristics similar to those of the A–4 and A–7 groups.

TABLE 4.—*Results of tests* [1] *of samples of soil from embankment and borrow pit areas in Gibson County, Indiana*

Location	Depth	Mechanical analysis						Physical constants			
		Particles larger than 2 mm.	Sand, 2 to 0.05 mm.	Silt, 0.05 to 0.005 mm.	Clay, smaller than 0.005 mm.	Colloids, smaller than 0.001 mm.	Passing No. 40 sieve	Liquid limit	Plasticity index	Shrinkage limit	Field moisture equivalent
	Feet		*Pct.*	*Pct.*	*Pct.*	*Pct.*					
Embankment area:											
Station 244	0–4	0	12	65	23	12	90	27	6	17	27
	4–9.5	0	8	65	27	10	99	27	8	17	27
	9.5–12	0	6	68	26	10	95	38	19	21	25
Station 268	0–3	0	9	60	31	15	99	28	11	18	23
	3–12	0	5	64	31	12	99	30	12	18	21
Cut from Station 238 to Station 241: Station 239	0–1	0	6	74	20	6	100	33	9	26	32
	1–18.5	0	5	42	53	15	100	33	13	18	26
Borrow pit A:											
Hole No. 2 [2]	0–1	0	5	65	30	14	100	31	13	19	26
	1–12	0	3	57	40	16	100	46	25	16	31
Hole No. 3 [3]	1–7	0	6	66	28	10	100	35	17	17	20
	10–14.5	0	2	60	38	14	100	39	17	19	26
Borrow pit B:											
Station 335	0–1	0	24	50	26	15	100	26	6	19	25
	1–6	0	8	53	39	18	100	36	16	19	30

[1] Tests performed in laboratory of State Highway Commission of Indiana.
[2] Station 3+00 on center line of borrow pit.
[3] Station 6+00 on center line of borrow pit.

Ohio Experiment.—The existing road on the Ohio project was surfaced and, therefore, borings were made with soil augers along the shoulders and ditches. Samples obtained from the different soil layers were tested in the laboratory of the State Highway Department at Columbus.

Four types of soils were found on the experimental section as shown on the soil profile in figure 4. The mechanical analyses and physical properties of these soils are given in table 5. Type 1 is a brown silty clay or clay having physical properties of the A–4 group. Type 2, a brown dense plastic clay, has physical characteristics similar to those of the group A–7 soils.

Type 1 is underlain in some locations by type 3, which is a gray, dense silty clay having physical properties of the A–4 group.

Type 4, a very heavy clay of the A–7 group, was found in making the borings but was not used in any of the embankments owing to its occurrence below the depth of excavation.

TABLE 5.—*Average results of tests performed on 37 samples of typical soil materials found in the subgrade survey in Delaware County, Ohio*

Soil type	Mechanical analysis					Physical constants		Number samples tested
	Particles larger than 2 mm.	Particles smaller than 2 mm.						
		Coarse sand, 2 to 0.25 mm.	Fine sand, 0.25 to 0.05 mm.	Silt, 0.05 to 0.005 mm.	Clay, smaller than 0.005 mm.	Liquid limit	Plasticity index	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*			
1	10	18	11	38	33	32	15	24
2	4	18	9	38	35	41	23	9
3	12	13	9	42	36	31	15	3
4	5	20	12	32	36	62	41	1

The soil in a borrow pit located about 1,000 feet to the right of station 18 was similar to the material found in the roadway cuts. Medium-sized boulders were found in all the soils.

MOISTURE-DENSITY CURVES PREPARED FOR USE IN CONTROLLING CONSTRUCTION OPERATIONS

In Indiana the maximum densities and optimum moisture contents of representative samples of the materials used in the embankment were determined by a compaction test similar to that described under method T 99–38 of the American Association of State Highway Officials. It differed from the standard method in that a separate portion of the sample was used for each change in moisture content.

Tests were performed in a field laboratory located at Buckskin. Results of these tests, table 6, were used to control the compaction of all sections and moisture content of the soil in section 1 during the construction of the fill. The results are shown graphically in figures 6 to 11.

During the construction of lifts 5 to 14 in section 2, lifts 5 to 14 in section 3, and the foundation layers in sections 4 and 5 of the Indiana project the power shovel was excavating material from a cut face extending from the surface to a depth of 8 feet in the part of borrow pit A east of the public road. The upper 4.5 feet at this location was represented by sample 2–A, and the lower 3.5 feet by sample 5–B. The average values of the maximum densities and optimum moisture contents of these two samples were used to control the compaction on the lifts when the material represented by these samples was placed.

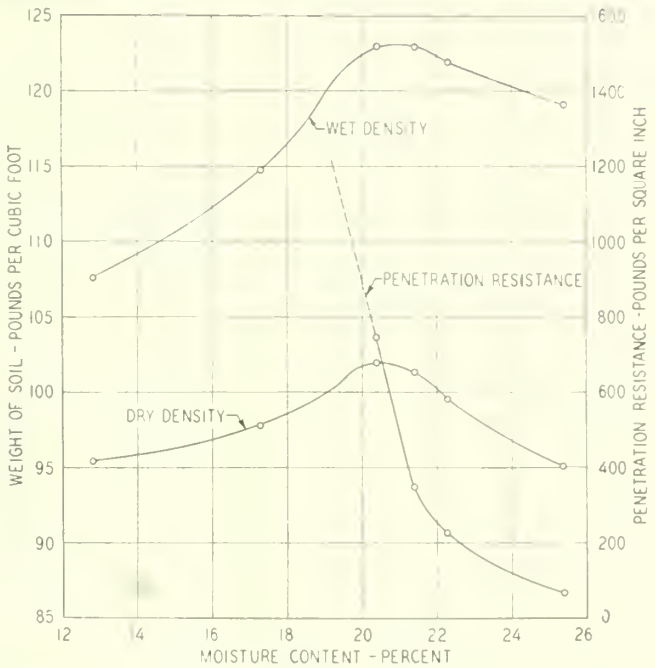
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 6.—DENSITY AND PENETRATION CURVES, SAMPLE 1–A IN INDIANA.

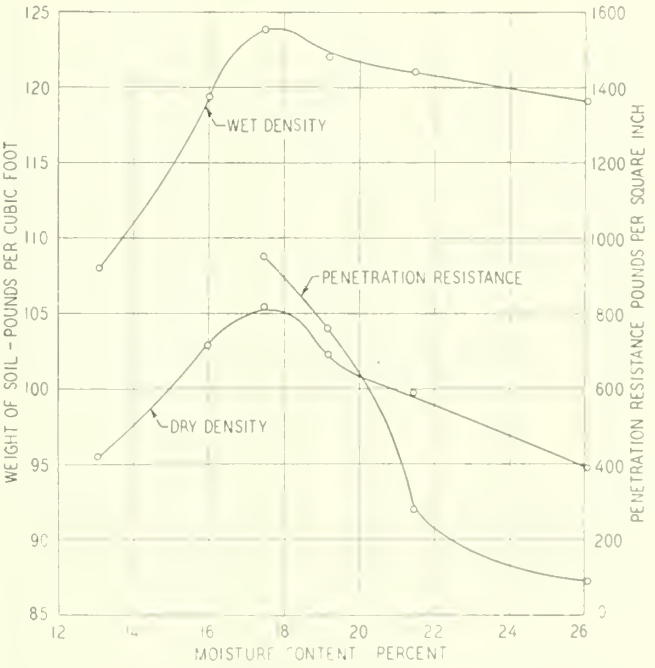
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 7.—DENSITY AND PENETRATION CURVES, SAMPLE 2–A IN INDIANA.

The average values were: Maximum wet density, 126.5 pounds per cubic foot; maximum dry density, 107.9 pounds per cubic foot; and optimum moisture content, 17.3 percent.

In Ohio the compaction tests were performed in the laboratory at Columbus and were in strict accordance with the standard method, A. A. S. H. O. designation T 99 38.

The results of the compaction tests performed on seven samples representative of the soils used in the

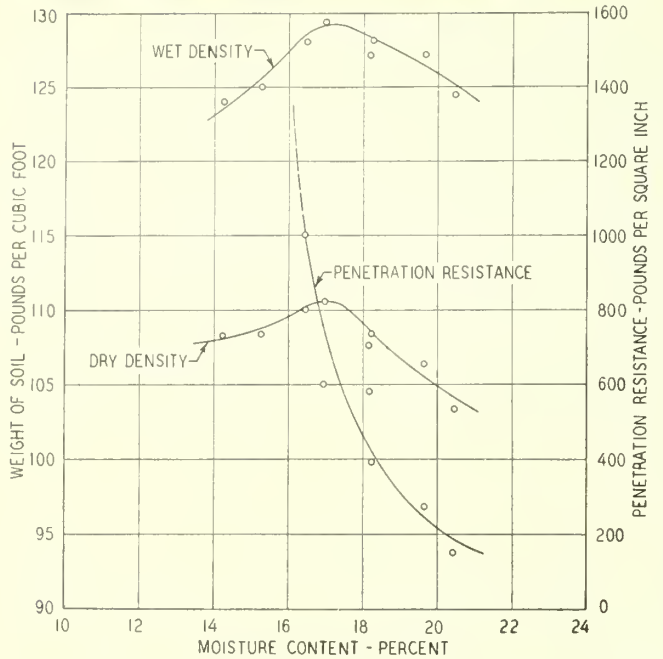
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 8.—DENSITY AND PENETRATION CURVES, SAMPLE 5–B IN INDIANA.

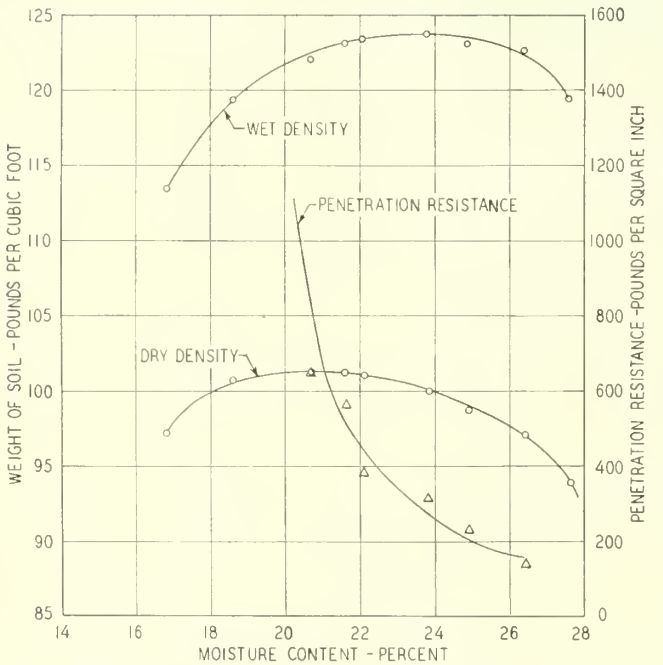
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 9.—DENSITY AND PENETRATION CURVES, SAMPLE 6 IN INDIANA.

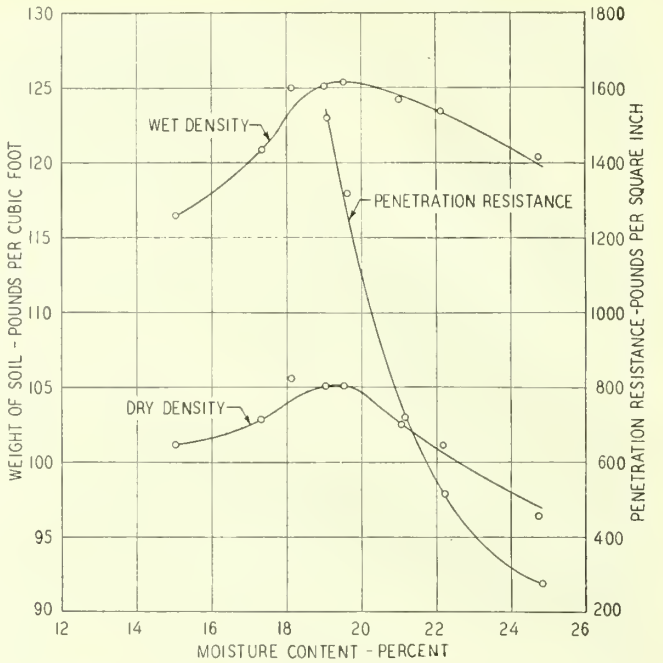
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 10.—DENSITY AND PENETRATION CURVES, SAMPLE 7–A IN INDIANA.

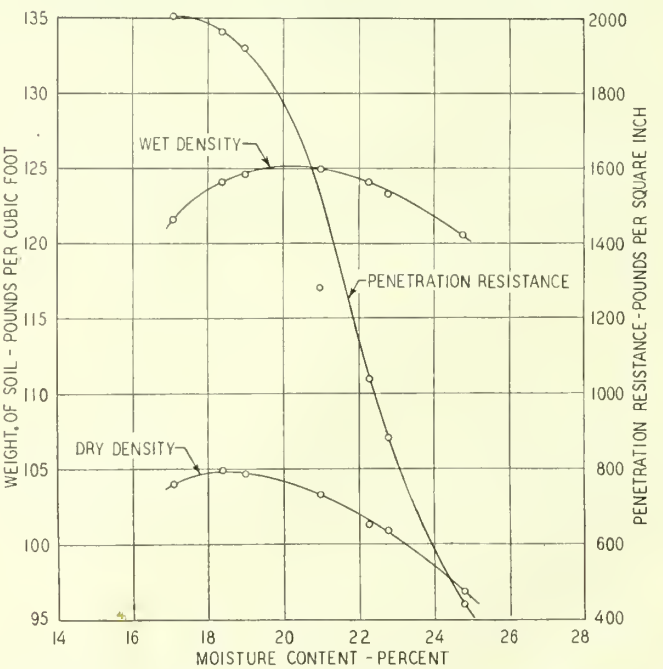
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 11.—DENSITY AND PENETRATION CURVES, SAMPLE 9–A IN INDIANA.

embankments in Ohio are shown graphically in figures 12 to 18 and together with their gradations, liquid limits, and plasticity indexes are given in table 7. These results are presented here in order to give a general idea of the character of the soils comprising the various fill sections.

A total of 81 samples were tested. The results were used to control the compaction and moisture content of the soil during the construction of the test sections.

CONSTRUCTION OF THE INDIANA PROJECT DESCRIBED

Foundation Layer.—Clearing of the embankment area in Indiana was completed during the first week of July. Construction of the fill was delayed until July 26 by heavy rains which flooded the low bottom lands with as much as 3 feet of water (fig. 1, *B*). After the water receded, the soil in the low area was so wet and soft that it would not support the construction equipment.

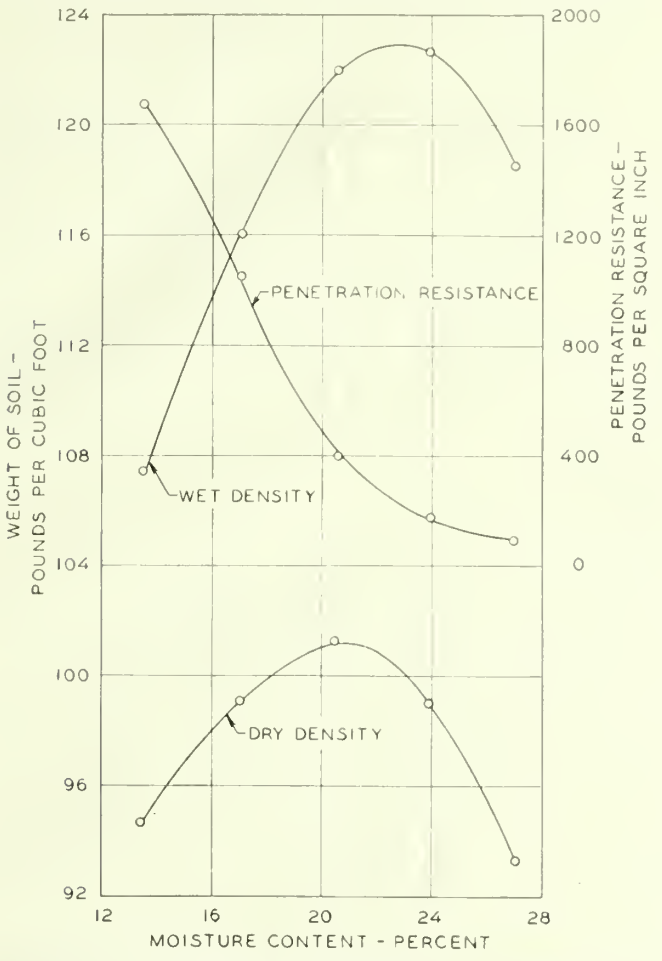

FIGURE 12.—DENSITY AND PENETRATION CURVES, SOIL NO. 13840 IN OHIO.

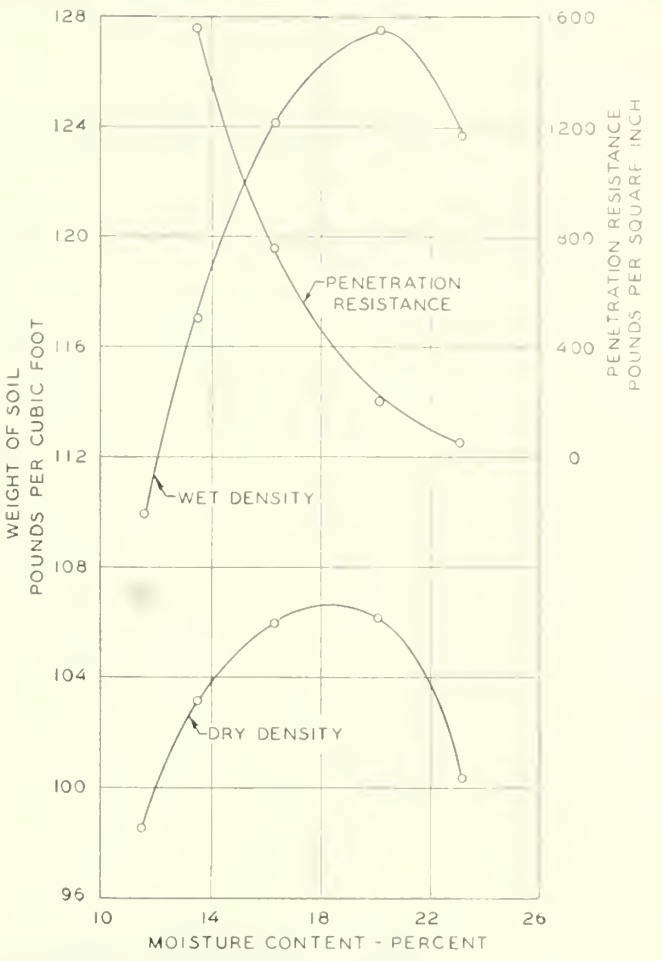

FIGURE 13.—DENSITY AND PENETRATION CURVES, SOIL NO. 14035 IN OHIO.

The condition of the soil where a tractor broke through is illustrated in figure 19.

As the soil dried very slowly a foundation layer was placed as shown in figure 3 to facilitate the operation of the equipment used in the construction of the experimental sections. The foundation layer was constructed to elevation 400 by end dumping from trucks, spreading the material with a bulldozer, and compacting with a sheepsfoot roller. The layer was placed in lifts having loose thicknesses of about 10 inches maximum.

TABLE 6.—*Results of compaction tests on embankment materials in Gibson County, Indiana*

Sample No.	Location	Depth	Maximum wet density	Maximum dry density	Optimum moisture content at maximum dry density	Resistance to penetration at optimum moisture content
		Feet	*Pounds per cubic foot*	*Pounds per cubic foot*	*Percent*	*Pounds per square inch*
1-A	Cut between station 238 and station 241.	0-4.5	123.0	102.0	20.8	580
2-A	Borrow pit A, east of public road.	0-4.5	123.9	105.3	17.6	940
5-B	Borrow pit A, east of public road.	4.5-8.0	129.2	110.5	17.0	730
6	Borrow pit A, east of public road.	8.0-15.0	123.7	101.2	20.6	520
7-A	Borrow pit A, west of public road.	0-5.0	125.2	105.0	19.6	1,280
9-A	Borrow pit B	0-5.5	125.0	104.8	18.4	1,960

Considerable difficulty was encountered in operating the equipment on the first lift of the foundation layer due to the surface irregularities produced by the subsidence and displacement of the soft undersoil. The sheepsfoot roller was very useful at this stage of the work since it would adapt itself to the uneven surface and could be backed out on soft soil which would not support the tractor. However, it was necessary to vary the amount of water in the drums to adjust the weight of the roller to fit the soft undersoil conditions.

Most of the foundation layer on sections 2 and 3 was placed with trucks having a capacity of 8 cubic yards. These were replaced by trucks holding 1.5 cubic yards which were found to be more satisfactory for the conditions existing on this job.

The foundation layer and the equipment used in its construction are shown in figure 20, *A*.

Construction Schedule.—Construction of sections 1 and 8 was commenced on September 24. The embankment area had dried out sufficiently by this time so that no foundation layer was required. However, the first two lifts on these sections were quite irregular and should be considered as a foundation layer.

The only road available for hauling material from borrow pit A to the embankment passed through section 1, making it necessary to delay the construction of this section until sections 2 to 7 were completed.

Construction of the fill proper started on sections 2 and 3, located between Pigeon Creek and the Old Wa-

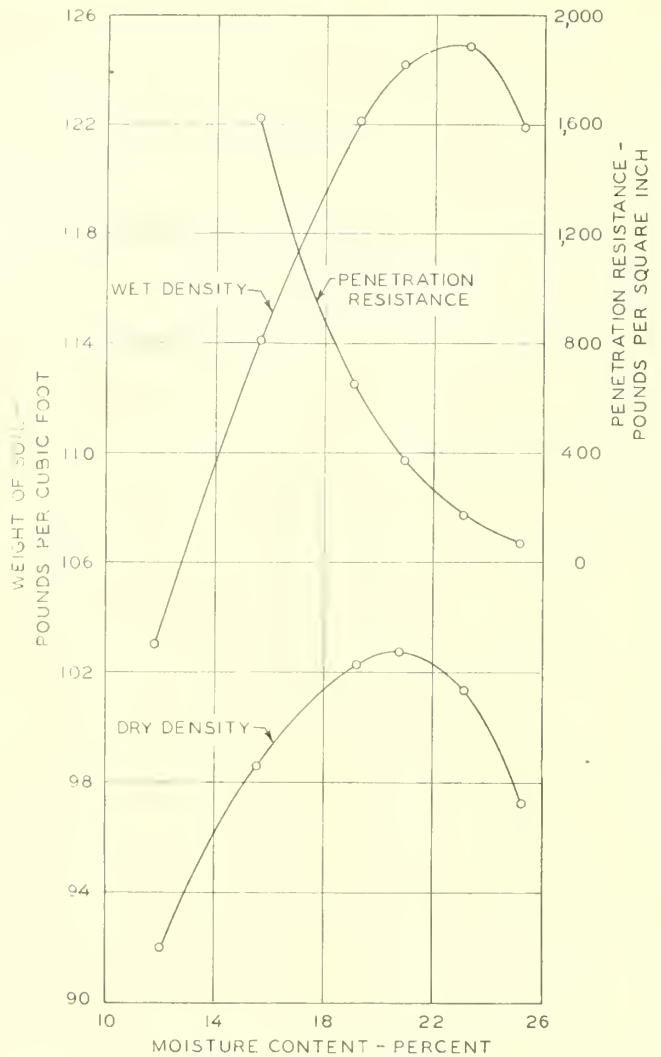
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 14.—DENSITY AND PENETRATION CURVES, SOIL NO. 14056 IN OHIO.

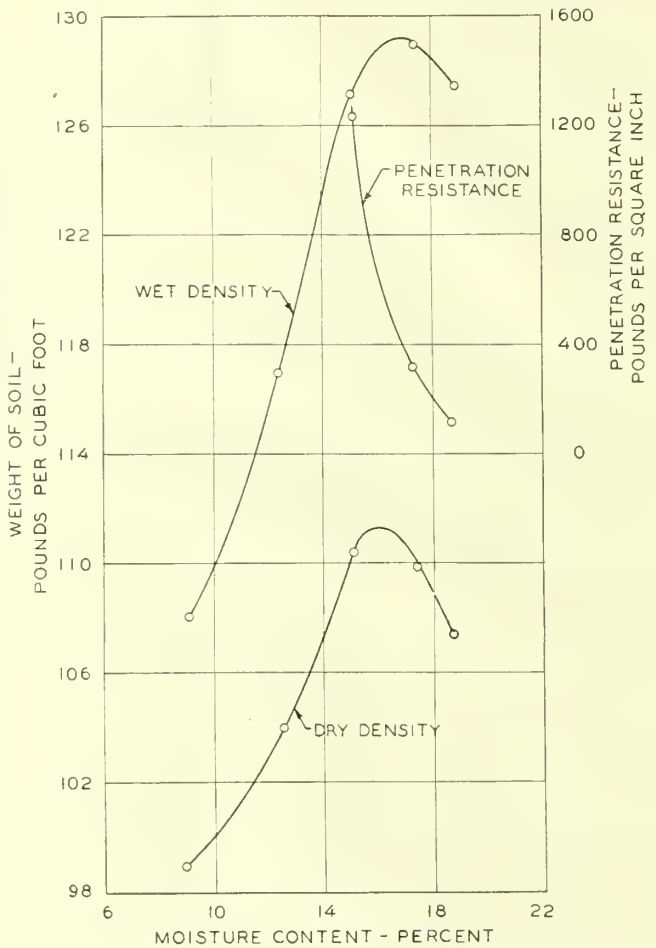
PENETRATION RESISTANCE
WET DENSITY
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 15.—DENSITY AND PENETRATION CURVES, SOIL NO. 14058 IN OHIO.

bash River and Erie Canal. Corresponding lifts on each section were constructed concurrently, the material for section 3 being hauled over section 2. Hauling of material and moving of equipment was not permitted on section 3 (compacted with sheepsfoot roller) during or subsequent to construction, except as necessary to construct the section.

The access road to sections 4, 5, 6, and 7 entered the fill area near the beginning of section 4. For this reason section 7 was constructed first, followed in order by sections 6, 5, and 4. Dates of construction of the different sections are given in table 8.

TABLE 8.—*Dates experimental sections were constructed in Gibson County, Indiana*

Section No.	Construction period in 1938			
	Foundation layer		Fill	
	Started	Completed	Started	Completed
1			Sept. 26	Oct. 12.
2	July 26	July 26	Aug. 10	Sept. 12.
3	July 28	July 28	Aug. 10	Sept. 17.
4	Aug. 13	Aug. 14	Sept. 9	Sept. 17.
5	Aug. 19	Aug. 21	Sept. 6	Sept. 9.
6	Aug. 21	Aug. 23	Sept. 2	Sept. 6.
7	Aug. 23	Aug. 24	Aug. 25	Sept. 2.
8			Sept. 24	Oct. 6.

TABLE 7.—*Results of tests performed on seven samples representative of soils used in embankments in Delaware County, Ohio*

Sample No.	Gradation					Liquid limit	Plasticity index	Maximum wet density	Maximum dry density	Optimum moisture content	Penetration resistance at optimum moisture content
	Passing 1-inch sieve	Passing No. 4 sieve	Passing No. 10 sieve	Passing No. 60 sieve	Passing No. 200 sieve						
	Percent	*Percent*	*Percent*	*Percent*	*Percent*			*Lb. per cu. ft.*	*Lb. per cu. ft.*	*Percent*	*Lb. per sq. in.*
13840		100	97	84	76	40	18	123.0	101.2	20.5	410
14035	100	99	97	89	81	39	17	127.6	106.6	18.6	400
14056	100	89	74	38	33	45	19	124.9	102.8	20.6	440
14058	100	95	75	18	16	37	16	129.2	111.3	15.8	800
14140		100	96	80	74	36	15	126.7	105.6	19.2	700
14147	100	94	92	75	65	32	14	129.6	110.8	16.0	610
14148	100	98	93	79	68	30	12	133.3	114.4	16.0	370

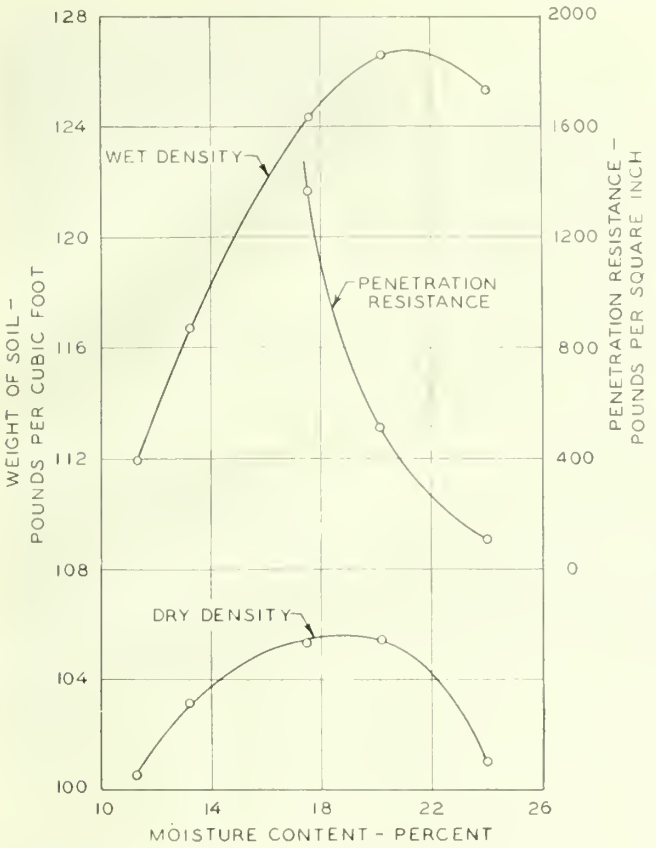
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 16.—DENSITY AND PENETRATION CURVES, SOIL NO. 14140 IN OHIO.

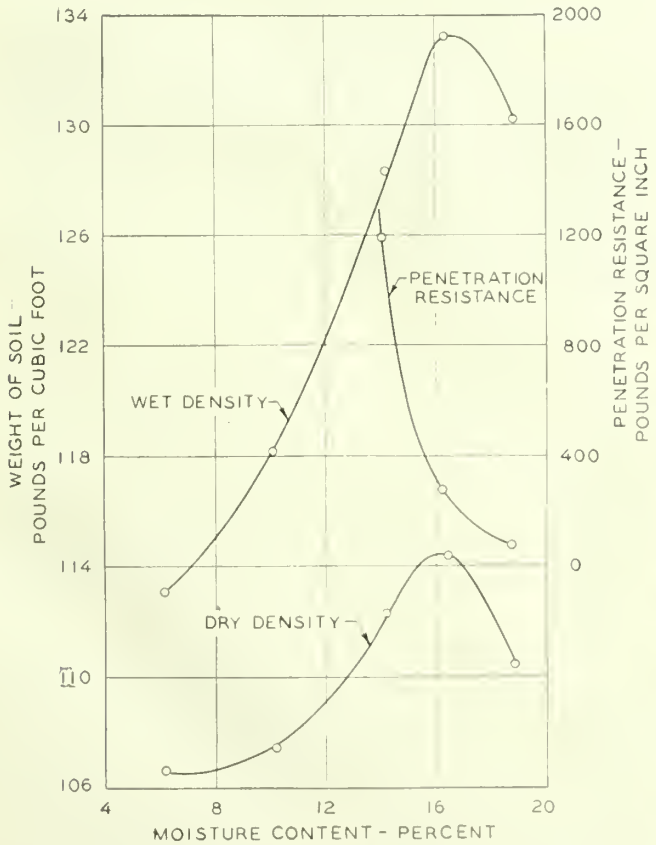
PENETRATION RESISTANCE
WET DENSITY
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 17.—DENSITY AND PENETRATION CURVES, SOIL NO. 14148 IN OHIO.

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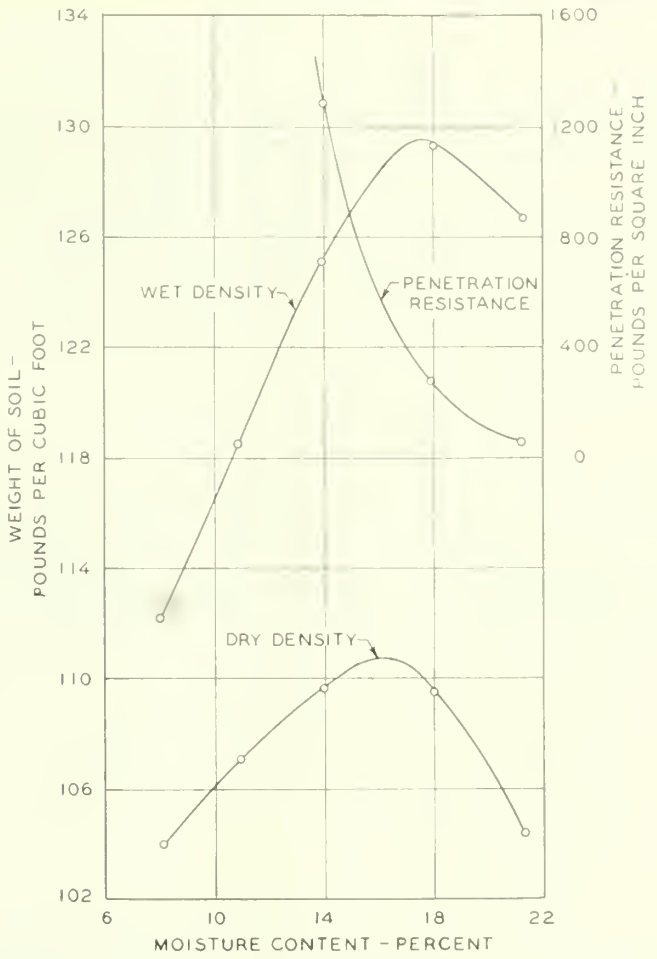
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE - POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT

FIGURE 18.—DENSITY AND PENETRATION CURVES, SOIL NO. 14147 IN OHIO.

Construction Procedure. Fill material for section 1 was placed in lifts 6 inches thick, loose measurement, by scrapers (fig. 20, *B*) having a capacity of about 8 cubic yards. The first two lifts were not uniform in thickness and compaction owing to the unevenness of the ground surface and the tendency of the fill material to be dis-

FIGURE 19.—SOFT CONDITION OF SOIL IN FILL AREA AFTER THE FLOOD WATERS RECEDED. A TRACTOR WAS MIRED AT THIS LOCATION IN INDIANA.

FIGURE 20.—CONSTRUCTION OF GRADE IN INDIANA SHOWING: (A) FOUNDATION LAYER OVER SOFT SOIL; (B) LARGE SCRAPER PLACING FILL MATERIAL ON SECTION 1; (C) COMPACTING WITH THREE-WHEEL 10-TON ROLLER; AND (D) COMPACTING WITH PNEUMATIC TIRE ROLLER.

placed as a result of the spongy character of the undersoil.

Following the placing of the first lift over the full width of the embankment area, each succeeding lift was constructed in two parts, each part being one-half the width of the fill section. One-half of the lift was compacted while material was placed on the adjoining half. This method was adopted in order to avoid compacting the spread material by any means other than by the roller specified for the section.

Prior to compaction, the soil on each half lift was tested to determine if its moisture content was within the tolerance of 1 of the optimum, as required by the specifications for this section. If the soil contained more than the specified amount, the soil was permitted to dry before compaction. A disk harrow was used to facilitate the drying. The disking served also to pulverize the clods of soil.

If the soil did not have sufficient moisture to comply with the specifications, water was added with a pressure distributor. Each application of water, approximately 0.5 gallon per square yard, was mixed with the soil by a disk harrow. Water was added and the disking continued until the required amount of moisture was uniformly distributed through the soil. The lift was then compacted with the sheepsfoot roller until it attained a density equal to at least 95 percent of the maximum as determined by the compaction test.

Fill material for sections 2 to 8 was excavated from the borrow pits by power shovels and hauled to the embankment in 1.5-ton trucks, spread with a bulldozer and road machine into layers of specified thickness, disked to pulverize the clods and loosen soil packed by equipment, and compacted to the required density by rolling. Moisture control was not used on these sections. When the soil was placed and compacted the soil had a moisture content as it came from the borrow pit. At times when the soil contained too much moisture to permit the operation of the compaction equipment, it was allowed to dry until the rollers could perform satisfactorily.

Section 3 like section 1 was compacted with a sheepsfoot roller; sections 2, 4, and 6 with the three-wheel roller (fig. 20, *C*); and sections 5, 7, and 8 with a pneumatic tire roller (fig. 20, *D*).

METHODS USED IN OHIO VERY SIMILAR TO THOSE IN INDIANA

Since the Ohio project was located over an old road it was necessary to remove the old road surfacing, the sod, and other objectionable material from the shoulders and side slopes before placing new fill. Figure 21, *A*, shows the old fill at the location of test section 4 cleared and ready for widening and raising of the grade. The widening was accomplished by placing material on each side of the old fill in horizontal layers of specified loose thickness. While the soil was being deposited on one side, the material on the other side was compacted.

After the widening had been completed, the material for each lift was placed for the full width of the roadway. Hauling was not permitted over any lift until it had been compacted to the required density.

Fill material for sections 1, 2, 3, 4, 8, and 9 was placed in lifts 6 inches thick, loose measurement, and in 9-inch lifts on sections 5, 6, and 7. Most of the fill material was excavated by power shovels hauled to the embankment in dump trucks (fig. 21, *B*) and spread with a bulldozer (fig. 21, *C*) into layers of specified

FIGURE 21.—TEST SECTION 4 IN OHIO SHOWING: (A) OLD FILL CLEARED AND READY FOR WIDENING AND RAISING OF THE GRADE; (B) HAULING WITH DUMP TRUCKS; (C) SPREADING WITH BULLDOZER; AND (D) HAULING AND SPREADING WITH SCRAPER.

FIGURE 22.—DISKING AND APPLYING WATER WITH A PRESSURE DISTRIBUTOR IN OHIO.

thickness. The excavation, hauling, and spreading of the soil used in some of the lifts on sections 1, 8, and 9 was done by 12-yard scrapers (fig. 21, *D*). The soil layer was then disked (fig. 22) to pulverize the clods.

Required moisture content was obtained in the same manner as described for section 1 of the Indiana project after which the lift was compacted to the required density. The pressure distributor used in Ohio is shown in figure 22.

DENSITY OF LIFTS DETERMINED

On both projects the number of trips that the particular roller was required to make in order to obtain the necessary compaction was governed by the specified density of the compacted lift.

Procedure in Indiana.—The density of the compacted lift in the Indiana experiment was determined in the following manner: The location where the density test was to be made was shaped with a shovel to give a level surface. A soil-collecting box, 18 inches square, having a 4.5-inch hole in the center was set in place on the leveled surface and a hole was bored to the bottom of the compacted layer with a 4-inch post-hole auger which passed through the hole in the soil-collecting box. This operation is illustrated in figure 23, *A*. The soil-collecting box served to collect the soil which spilled through the sides and over the top of the auger while making the boring. This material, together with the soil removed by the auger, was placed in a bucket and weighed. The weight of the bucket, subtracted from the weight of bucket and soil, gave the weight of the soil removed from the hole. For determination of the moisture content a representative sample of about 80 gm. was placed in a container and sealed. The moisture content was later determined in the field laboratory at Buckskin.

Standard Ottawa sand of known weight per cubic foot, loose measurement, was poured into the hole from a canvas sack. The combined weight of the sack and sand was determined before placing any sand in the hole. When the hole was filled to the top of the lift

FIGURE 23.—REMOVING SOIL WITH POST-HOLE AUGER TO DETERMINE DENSITY OF COMPACTED LIFT AND DETERMINING THE VOLUME BY FILLING THE HOLE WITH SAND OF KNOWN WEIGHT PER UNIT VOLUME (INDIANA PROJECT).

(fig. 23, *B*), the sack and the sand remaining was weighed. The difference between the two weights gave the weight of sand in the hole.

All weighing was done with a balance having a capacity of about 30 pounds and accurate to 0.5 ounce.

The weight of sand required to fill the hole was divided by its weight per cubic foot to determine the volume of the hole. The weight of the soil removed from the hole was then divided by the volume of the hole to obtain the wet density of the compacted lift in pounds per cubic foot.

The dry density of the compacted lift in pounds per cubic foot was calculated by the formula:

$$\text{Dry density} = \frac{\text{wet density} \times 100}{100 + \text{percentage of moisture in soil}}$$

Determination of Dry-Sand Volume.—The weight per cubic foot of the loose dry Ottawa sand used in the density tests was calculated in the following manner: A container of known volume having approximately the same dimensions as the auger hole was filled with sand poured from a canvas bag. The mouth of the bag was constricted in such a manner that the sand was discharged in a loose stream having a diameter of about 1.5 inches. The bag was raised as the container filled in order to provide a constant height of fall of 12 inches.

The difference in the weights of the container, empty and filled with sand, equals the weight of sand in the container. This weight of sand divided by the volume of the container gives the density of the loose dry sand in pounds per cubic foot.

During the operation care was exercised to avoid vibration of the container which might cause the sand to settle.

The procedure used in pouring the sand into the container for determination of its weight per cubic foot was followed in placing the sand in the auger holes.

Procedure in Ohio.—With several exceptions the method used in Ohio was similar to that used in Indiana. Only the exceptions need to be noted.

Instead of the collecting box 18 inches square, a pan 14 inches square with a 4.5-inch circular opening in the center was used. After the sample had been collected, a metal cover was fitted over the opening in the pan in order to prevent the loss of soil, the pan and soil were

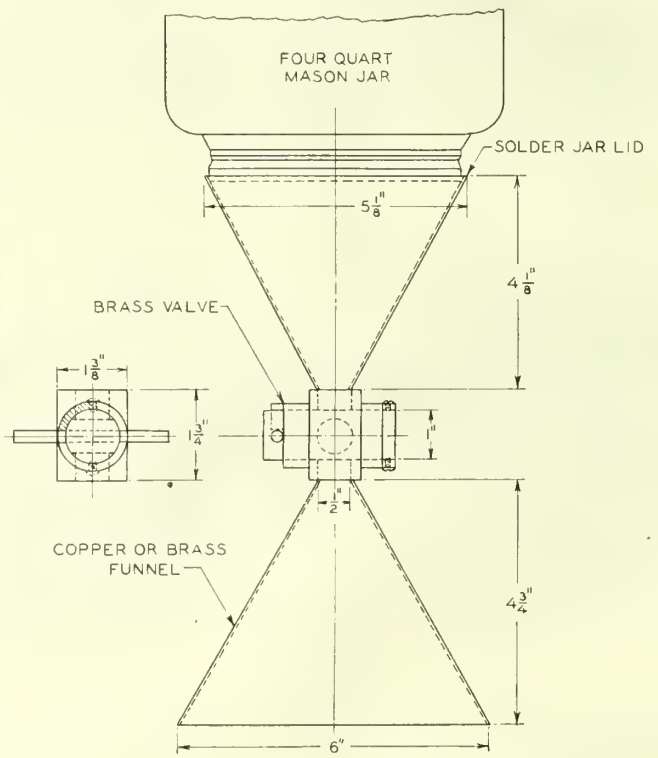

FIGURE 24.—DETAILS OF JAR-CONE DEVICE USED IN MAKING DENSITY TEST IN DELAWARE COUNTY, OHIO.

weighed, and the weight of the soil was calculated by subtracting the known weight of the pan from the combined weight of the pan and soil.

Standard Ottawa sand of known weight per cubic foot, loose measurement, was poured into the hole from a glass jar fitted with a detachable apparatus consisting of a metal cone, a shut-off valve, and a funnel as shown in figure 24. The combined weight of this jar-cone apparatus and the sand contained therein was determined before placing sand in the hole. The jar-cone apparatus was placed in an inverted position over the hole (see cover illustration) and the valve opened. The sand flowed until the hole and funnel were completely filled. The valve was closed and jar-cone apparatus with the sand remaining in it weighed.

The difference between the two weights gave the weight of the sand in the hole and funnel. By subtracting the constant weight of the sand in the funnel, the weight of the sand required to fill the hole was found. All weighing was done with a balance having a capacity of about 35 pounds and accurate to 0.01 pound.

Calibration of Sand.—Before conducting any tests on the embankment, the dried sand was calibrated for weight per cubic foot by placing the jar-cone apparatus, with funnel up and valve closed, on a vibrationless support. The funnel was then filled and the valve opened. As the sand flowed into the jar more sand was poured into the upper funnel, maintaining the sand in the funnel level with the top at all times. When the jar and cone were full, the valve was closed and the excess sand poured from the funnel.

The weight of sand contained in the jar and cone was determined by obtaining the difference in weight of the full and empty apparatus. This weight divided by the volume of the apparatus, previously determined with water, gave the weight per cubic foot of the sand under standard flow conditions.

To calibrate the constant weight of dry sand contained in the 6-inch funnel, the weighed apparatus was inverted on a smooth surface and sand allowed to flow into the funnel until full. The difference between the weight of the apparatus with the remaining sand and the weight of the apparatus full of sand gave the weight of the sand in the funnel.

Correction for Coarse Material.—The standard compaction test used in the laboratory showed the maximum density of that portion of the soil which passed the No. 4 sieve. Inclusion of material larger than the No. 4 sieve would have the effect of increasing this density owing to the higher specific gravity of the stone, approximately 2.65, as compared with the bulk specific gravity of the compacted soil, about 1.76 for soil having an average dry density of 110 pounds per cubic foot. Lifts, compacted to specified densities based on the laboratory tests, and which contain material retained on the No. 4 sieve, will have densities higher than those indicated by the standard test.

In this case the specified densities applied to the material passing the No. 4 sieve and consequently, a correction was made for the effect of the material retained on the No. 4 sieve. This correction was made in the following manner: Instead of using the total weight of soil extracted from the auger hole in computing the density of a lift, a value equal to this weight minus one-third of the weight of the material retained on the No. 4 sieve was used (ratio of specific gravities given above is 3 to 2).

All field tests in Ohio were performed in a portable laboratory (fig. 25) stationed on the job.

FIGURE 25.—PORTABLE FIELD LABORATORY IN OHIO.

CONTROL OF THE MOISTURE CONTENT ON INDIANA PROJECT

On sections 2 to 8 where moisture control was not specified records of the moisture content of each lift were obtained as a part of the density determination previously described.

Two samples were taken from each half lift on section 1, and the moisture contents of the samples were determined by drying them in pans over an open flame, care being taken to constantly stir the soil to prevent burning. The sample (about 80 gm.) was dried in approximately 10 minutes. The weighings were made on a triple beam balance having a capacity of 111 gm. and sensitive to 0.01 gm.

Additional samples were taken from locations where visual inspection indicated important differences in moisture content.

When the soil of section 1 needed either wetting or drying to give it the specified moisture content, moisture determinations were made after each application of water or during the disking to expedite evaporation. It was seldom necessary to dry the soil placed in section 1. Of the 20 lifts comprising the fill 17 required the addition of water in amounts ranging from 0.5 to 3 gallons per square yard.

COMPACTION CONTROL AND DENSITY DETERMINATIONS ON OHIO PROJECT

Use of Typical Curves.—The soil delivered to the fills on the Ohio project was obtained from the cuts and borrow pits in such a manner that it was a mixture of the materials represented by the samples for which moisture-density curves were determined. It was decided to adopt the method previously used with satisfactory results in Ohio [5] when such variations were found. Selection was made from the group of curves available as a result of the tests made on samples from different parts of the cuts and borrow pits, the one most representative of the material being placed. Selection was made in the following manner: The soil to be used in the fill was compacted in the compaction cylinder according to the standard procedure and its wet weight per cubic foot was determined. Penetration by the Proctor plasticity needle then disclosed its penetration resistance

[5] SOIL MECHANICS APPLIED TO HIGHWAY ENGINEERING IN OHIO. Engineering Experiment Station Bulletin 99, Ohio State University, July 1938.

in pounds per square inch. The observed wet density corresponds to a range of moisture contents on the group of moisture-density curves and the observed penetration resistance corresponds to a range of moisture contents on the group of moisture-penetration-resistance curves. Using the observed penetration and density the moisture-density and the moisture-penetration curves for each sample were examined and the corresponding moisture determined on each curve. The soil for which these two values were most nearly equal was considered to be representative of the soil being placed and its moisture-density curve was used in further determinations.

Moisture Control.—The moisture determinations were made as the soil was delivered on the fill and after the various applications of water.

The typical moisture-density and moisture-penetration-resistance curves having been selected for a soil to be used in a particular lift, the density determinations were made in the usual manner. For checking the moisture content of the compacted material the Proctor plasticity needle, which disclosed the penetration resistance of the compacted lift, was used (see cover illustration). The moisture content of the soil in the fill was taken as the moisture content corresponding to the observed density on the typical moisture-density curve. This value was checked by using the observed penetration to determine moisture on the moisture-penetration curve.

Compaction Control.—As in the case of moisture control, the compaction was controlled in accordance with the moisture-density curves selected as described above. Table 9 shows that as many as 22 different curves were used on a section. The average maximum density determined from all curves used in each section, together with the range in maximum densities and the variations in compaction obtained in the fills are given in table 10. The table shows that compaction in excess of 100 percent of maximum was obtained on parts of every section. This indicates that actual field compaction, at places, exceeded that of the standard compaction test.

TABLE 9.—*Variation between moisture content of compacted fill and optimum in Delaware County, Ohio*

Section No.	Optimum moisture content [1]			Variation from optimum			Lifts below optimum [3]	Moisture-density curves
	Average [2]	Minimum	Maximum	Average	Minimum	Maximum		
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Number*
1	18.8	14.2	22.0	−2.0	−0.8	−4.9	94	12
2	18.2	15.5	22.0	−2.2	−.3	−4.2	96	16
3	17.6	14.4	20.6	−2.0	0	−4.8	97	17
4	18.4	15.2	22.0	−1.0	+.1	−4.0	74	18
5	20.0	16.0	23.0	−2.0	−.9	−4.4	96	11
6	16.6	15.3	18.2	−.8	−.4	−3.5	83	10
7	18.7	16.0	23.0	−.5	−.3	−1.7	80	5
8	17.7	12.7	23.0	−1.8	−.1	−5.5	95	22
9	17.6	14.7	20.2	−2.0	0	−5.2	94	11

[1] Determined in laboratory by means of standard compaction test.
[2] Average from curves used in test section.
[3] Percentage of lifts compacted at moisture contents below optimum.

TABLE 10.—*Variation in densities in Delaware County, Ohio*

Section No.	Maximum dry densities [1]			Compaction [3]				Moisture-density curves
	Average [2]	Minimum	Maximum	Average	Minimum	Maximum	Specified	
	Lb. per cu. ft.	*Lb. per cu. ft.*	*Lb. per cu. ft.*	*Percent*	*Percent*	*Percent*	*Percent*	*Number*
1	106.1	102.4	111.3	99.5	91.0	108.5	100	12
2	108.3	102.4	113.7	105.2	93.5	113.7	100	16
3	109.4	102.7	114.5	102.9	93.9	111.9	100	17
4	107.4	102.4	113.7	101.5	94.4	106.9	100	18
5	105.5	102.3	111.7	104.2	96.5	111.4	100	11
6	110.3	107.0	114.4	97.1	92.5	100.7	100	10
7	107.2	102.3	111.7	101.4	100.0	102.5	100	5
8	109.4	99.2	118.7	97.3	83.0	107.5	95	22
9	109.3	103.1	113.5	101.1	91.6	108.6	95	11

[1] Determined in laboratory by means of standard compaction test.
[2] Average for curves used in test section.
[3] Percentage of maximum dry density obtained in the compacted fill.

DETAILED RECORD MADE OF CONSTRUCTION OPERATIONS

In view of the experimental nature of the construction, an attempt was made to control the thickness of the fill layers as accurately as possible. The loose thickness of each lift was measured by taking elevations at the same spots before and after spreading soil. The difference in elevation was taken as the loose thickness of lift.

In Indiana the measurements were made at locations 50 feet apart along lines through the quarter points of the lift. If the measured thickness differed from the specified thickness by more than 0.1 foot for a distance of 50 feet, material was added or bladed off as required. An engineer's level was used in taking elevations. All measurements in the Ohio experiment were made in a particular area of the fill section about 15 feet to the right of the center line at the station where the fill was a maximum in height and where the settlement cells, described later in this report, were placed.

In order to obtain data relative to the uniformity of the embankment and the performance of the different types of compacting equipment, the following records were made during the construction of each section on both projects:

1. Type of roller used for compaction.
2. Maximum density and optimum moisture content of the material comprising each lift.
3. Number of lifts.
4. Loose thickness of each lift.
5. Moisture content of soil in each lift.
6. Amount of water added to fill material.
7. Number of trips with roller to obtain specified density.
8. Thickness of lift after compaction.
9. Density of compacted lift.
10. Moisture content of compacted lift.

The following time studies were made to obtain information on the efficiency of the different rollers as indicated by the production in volume of material compacted per hour of rolling time:

1. Total available working time for section.
2. Actual working time of rollers.
3. Delays due to weather conditions, equipment repairs, waiting for material, and other operations.
4. Operating speed of rollers and other equipment.
5. Time required for various construction operations in connection with moisture control on section 1 of the Indiana project and on all sections of the Ohio project.

A summary of the construction records relating to the amount and percentage of compaction, the moisture contents of the compacted lifts, and the amount of rolling required to obtain the desired density are given in table 11 for the Indiana work, and in table 12 for the Ohio work. The results of the time and production studies presented in tables 13 and 14 were summarized from data furnished by H. L. Arbenz and W. R. Lytz, respectively, of the Division of Construction, Public Roads Administration.

TABLE 11.—*Construction record summary in Gibson County, Indiana*

Section No.	Type roller	Number lifts	Loose thickness of lifts specified	Average thickness of lifts constructed		Moisture content				Wet density		Percentage of maximum wet density		Number trips with roller to obtain required density			Remarks
				Loose	Compacted	Optimum [1]	Compacted fill			Maximum [2]	Average, compacted fill	Required	Average obtained	Average	Minimum	Maximum	
							Average	Minimum	Maximum								
			Feet	*Feet*	*Feet*	*Percent*	*Percent*	*Percent*	*Percent*	*Pounds per cubic foot*	*Pounds per cubic foot*	*Percent*	*Percent*				
1	Sheepsfoot	2															Foundation layer
		1	0.50	0.56	0.54	21.6	23.3	22.2	24.4	123.7	124.1	95	100.3	4.5			Lift 11
		17	.50	.50	.48	19.6	19.5	16.5	22.4	125.2	122.6	95	97.9	8.0	6	18	Lifts 3 to 10 and 12 to 20.
2	3-wheel	4															Foundation layer
		10	.50		.50	17.3	22.9	20.8	24.7	126.5	123.3	95	97.5	2.1	2	3	Lifts 5 to 14.
		4	.50	.56	.48	21.6	24.8	24.6	27.9	123.7	123.9	95	100.2	2.0	2	2	Lifts 15 to 18.
		2	.50		.46	19.6	20.8	20.5	21.2	125.2	124.5	95	99.5	1.8	1	2	Lifts 19 and 20.
3	Sheepsfoot	1															Foundation layer
		10	.50		.51	17.3	22.4	21.2	24.6	126.5	121.8	95	96.3	7.8	6	13	Lifts 5 to 14.
		2	.50	.52	.44	21.6	26.6	24.5	28.7	123.7	118.1	95	95.5	7.2	5	12	Lifts 15 and 16.
		2	.50		.42	19.6	20.4	20.1	20.7	125.2	120.4	95	96.2	5.5	5	6	Lifts 17 and 18.
4	3-wheel	3															Foundation layer.
		1	.75			21.6	20.4			123.7	128.2	95	103.6	2.0			Lift 1.
		6	.75	.77	.72	19.6	21.0	17.1	25.3	125.2	126.7	95	101.2	2.5	2	5	Lifts 5 to 10.
5	Pneumatic tire	3															Foundation layer
		8	.75	.76	.69	21.6	22.9	20.2	25.7	123.7	120.5	95	97.4	3.9	3	12	Lifts 4 to 11.
6	3-wheel	2															Foundation layer
		7	1.00	.95	.87	21.6	24.3	21.5	26.3	123.7	123.6	95	99.9	2.0	2	2	Lifts 3 to 9.
7	Pneumatic tire	2															Foundation layer.
		8	1.00	.99	.84	21.6	27.1	23.0	29.5	123.7	120.4	95	97.3	2.7	2	3	Lifts 3 to 10.
8	Pneumatic tire	2															Foundation layer.
		13	.50	.55	.52	18.4	20.4	14.9	26.3	125.0	123.7	95	99.0	2.9	2	6	Lifts 3 to 15.

[1] Determined from compaction tests performed in laboratory.
[2] Determined from compaction tests performed in laboratory. Figures given are maximum values for the compaction method used in the test and not absolute maximums.

TABLE 12.—*Construction record summary in Delaware County, Ohio*

Section No.	Type roller	Number lifts	Thickness loose material specified per lift	Average thickness per lift constructed		Average moisture content		Average dry density		Maximum dry density		Number trips with roller to obtain required density			Number moisture-density curves
				Loose	Compacted	Optimum [1]	Compacted fill	Maximum [1]	Compacted fill	Required	Average obtained	Average	Minimum	Maximum	
			Feet	*Feet*	*Feet*	*Percent*	*Percent*	*Pounds per cubic foot*	*Pounds per cubic foot*	*Percent*	*Percent*				
1	Sheepsfoot	21	0.50	0.54	0.42	18.8	16.8	106.1	105.6	100	99.5	9.1	8	12	12
2	3-wheel	31	.50	.61	.49	18.2	16.0	108.3	113.9	100	105.2	2.6	2	4	16
3	Pneumatic tire	32	.50	.51	.39	17.6	15.6	109.4	112.6	100	102.9	4.8	3	6	17
4	Sheepsfoot	28	.50	.54	.38	18.4	17.4	107.4	109.0	100	101.5	6.3	5	10	18
5	3-wheel	28	.75	.66	.54	20.0	18.0	105.5	109.9	100	101.2	4.1	2	8	11
6	Sheepsfoot	13	.75	.78	.62	16.6	15.8	110.3	107.1	100	[2] 97.1	6.0	6	6	10
7	Pneumatic tire	10	.75	.90	.68	18.7	18.2	107.2	108.7	100	101.4	6.0	4	8	5
8	Sheepsfoot	38	.50	.51	.40	17.7	15.8	109.1	106.1	95	97.3	8.1	3	20	22
9	3-wheel	35	.50			17.6	15.6	109.3	110.5	95	101.1	3.3	1	8	11

[1] Average of all optimum moisture contents and maximum densities as determined in laboratory on the different soils placed in the test section.
[2] Two types of sheepsfoot rollers used on this section. All lifts compacted with 6 roller trips and densities recorded for comparison of compaction produced by each roller.

CONTROL OF MOISTURE CONTENT WITHIN A NARROW RANGE FOUND TO BE PRACTICABLE

Indiana Project.—As shown in table 11 the average moisture content of 17 of the 20 lifts in section 1 was only 0.1 below the optimum for the material used. On one lift the average was 1.7 higher than the optimum. Although in a few instances the variation from the optimum was somewhat greater than the 1 specified, it is evident that the moisture control practiced on this section produced a high degree of uniformity in moisture content. Variations from the optimum were found to be greater on the other sections where the fill material was placed as it came from the borrow pit without regard to its moisture content.

Ohio Project.—It was not practicable to keep the moisture content within the range of 1 of the optimum value. An attempt was made to maintain the moisture within the specified limits, but wider variations were permitted where satisfactory compaction was obtained and rigid control of the moisture content would have affected the progress of the work to such an extent as to be impracticable.

Variations between the average moisture contents of the compacted fills and the optimum moisture contents of the soils are given in table 9. The average variation for all the sections was 1.7 below the optimum, while average variation for the different sections ranged from 0.5 to 2.2 below the optimum. Although the variations in individual cases were somewhat higher, reaching a maximum of 5.5 below the optimum in section 8, the average values indicate that the moisture content of the soil can be maintained reasonably close to the optimum.

It is of interest to observe that on most of the work the compacted soil did not have a moisture content greater than the optimum. The number of lifts compacted at moisture contents below the optimum amounted to 92 percent on the nine test sections and varied from 74

percent on section 4 to 97 percent on section 3. This condition occurred in spite of the addition of water to all but a few of the lifts placed. The additional moisture was not sufficient, in most cases, to raise the moisture content to the optimum.

WEATHER INFLUENCED THE MOISTURE CONTENT OF FILLS IN INDIANA

The optimum moisture contents of the different soil materials used in the fill, the average variations from the optimum, and minimum and maximum variation from the optimum of samples of compacted soil are given in table 15, together with dates of construction of the various lifts and the sources of the fill material.

A study of table 15 and the weather records indicate that, in the absence of moisture control, the moisture content of the compacted lifts was influenced largely by weather conditions. The variation from the optimum was greatest in the portions of sections 2, 3, and 7 constructed in August, when there was a considerable amount of rain. In most cases the moisture content was higher than the optimum which was to be expected since in addition to the August rains the rains in July had been so heavy and frequent that practically no construction work could be performed. In contrast relatively small variations were found on sections 4, 5, and 6; lifts 19 and 20 of section 2; and lifts 17 and 18 of section 3 placed during September 2 to 17.

MOISTURE CONTROL DOES NOT CAUSE SERIOUS DELAYS IN CONSTRUCTION

Indiana Project.—One of the major delays in the construction of fills was waiting for material. This was due to an insufficient number of hauling units. Hauling operations were never delayed because of rolling. The time lost waiting for material (table 13) was greater than the actual rolling time required to compact the fill. Each of the rollers readily compacted all of the material delivered to the fill.

The hauling equipment was delayed on section 1 to correct the moisture content of the soil. Sprinkling of dry soil, drying of wet soil, and necessary disking resulted in a delay of 25.46 equipment hours. Addition of water by a distributor operating at a speed of about 3 feet per second took an average of 25 minutes per lift although as much as an hour was required on some lifts. Mixing of the soil and water was done with a disk harrow which operated at an average speed of 200 feet per minute. The average time for disking was 35 minutes per lift, but this varied according to the width of the lift and amount of water added.

In only one instance was it necessary to reduce the moisture content of the soil delivered to the fill prior to compaction. The soil was spread and left to dry with occasional disking to facilitate the drying. Six hours of intermittent drying and disking were required to process the soil in this manner.

Delays resulting from the moisture control occurred only on the first six lifts on section 1. As the various operations of wetting and disking the fill material became more familiar to the construction crew, delays were avoided.

Ohio Project.—The actual number of hours that the rollers were in operation was a small percentage of the time that they were on the job. Most of the time was consumed in waiting for material and for other operations in connection with hauling, spreading, disking, and sprinkling. Construction work was never delayed because of the rolling. Each of the rollers readily compacted all of the material delivered to the fills.

It was necessary to add water to almost all lifts in order to comply with specifications for moisture control. This was done with an 800-gallon pressure dis-

TABLE 13.—*Summary of time and production studies of rolling in Gibson County, Indiana*

Section No.	Type roller	Thickness loose material per lift	Average speed of roller	Use of available working time			Volume compacted		Rolling time per cubic yard compacted	Average roller trips to obtain required density
				Rolling [1]	Waiting for material	Other operations	Total for section	Per hour of rolling		
		In.	*Ft. per min.*	*Hr.*	*Hr.*	*Hr.*	*Cu. yd.*	*Cu. yd.*	*Sec.*	
1	Sheepsfoot	6	256	32.97	41.98	97.05	11,250	341	10.55	7.8
2	3-wheel	6	160	25.28	35.78	9.94	6,150	244	14.80	2.0
3	Sheepsfoot	6	185	19.08	26.85	5.07	4,855	255	14.11	7.4
4	3-wheel	9	150	11.08	15.67	9.25	5,074	458	7.85	2.4
5	Pneumatic tire	9	175	12.40	13.60	10.00	5,432	438	8.21	3.9
6	3-wheel	12	144	8.73	17.17	9.10	4,974	570	6.33	2.0
7	Pneumatic tire	12	220	11.27	29.08	6.40	6,280	557	6.44	2.7
8	Pneumatic tire	6	250	18.60	52.27	23.48	6,720	361	9.97	2.9

[1] Actual number of hours rollers were used for compacting.

TABLE 14.—*Summary of time and production studies in Delaware County, Ohio*

Section No.	Type roller	Thickness loose material per lift	Average speed of roller	Use of available working time			Volume compacted		Rolling time per cubic yard compacted	Average roller trips to obtain required density
				Rolling [1]	Waiting for material	Other operations	Total for section	Per hour of rolling		
		In.	*Ft. per min.*	*Hr.*	*Hr.*	*Hr.*	*Cu. yd.*	*Cu. yd.*	*Sec.*	
1	Sheepsfoot	6	270	15.43	1.92	15.39	2,794	181	19.9	9.1
2	3-wheel	6	210	21.94	------	66.59	3,967	181	19.9	2.6
3	Pneumatic tire	6	280	41.74	2.33	41.28	9,159	220	16.4	4.8
4	Sheepsfoot	6	240	29.29	25.27	34.74	8,156	279	12.9	6.3
5	3-wheel	9	295	25.60	11.06	29.38	6,315	247	14.6	4.1
6	Sheepsfoot	9	320	8.73	2.50	6.10	2,232	255	14.1	6.0
7	Pneumatic tire	9	280	6.15	10.25	13.99	1,705	277	13.0	6.0

[1] Actual number of hours the rollers were used for compacting.

TABLE 15.—*Dates of placing material from different sources and variations from optimum moisture content in Gibson County, Indiana*

Section No.	Lift No.	Source of fill material		Optimum moisture content	Variation from optimum			Construction dates
		Borrow pit	Depth below surface		Average	Minimum	Maximum	
			Feet	*Percent*	*Percent*	*Percent*	*Percent*	
1	11	A	8–15	21.6	[1]+1.7	+0.6	+2.8	Oct. 4.
	3 to 10	A	0–5	19.6	−.1	−3.1	+2.8	Sept. 28–Oct. 4.
	12 to 20	A	0–5	19.6	−.1	−3.1	+2.8	Oct. 5–12.
2	5 to 14	A	0–8	17.3	+5.4	+3.5	+7.4	Aug. 10–19.
	15 to 18	A	8–15	21.6	+3.2	+3.0	+6.3	Aug. 30–Sept. 1.
	19 to 20	A	0–5	19.6	+1.2	+.9	+1.6	Sept. 12.
3	5 to 14	A	0–8	17.3	+5.1	+3.9	+7.3	Aug. 10–19.
	15 to 16	A	8–15	21.6	+5.0	+2.9	+7.1	Aug. 30.
	17 to 18	A	0–5	19.6	+.8	+.5	+1.1	Sept. 12 and 17.
4	4	A	8–15	21.6	−1.2	------	------	Sept. 9.
	5 to 10	A	0–5	19.6	+1.4	−2.5	+5.7	Sept. 9–17.
5	4 to 11	A	8–15	21.6	+1.3	−1.4	+4.1	Sept. 6–9.
6	3 to 9	A	8–15	21.6	+2.7	−.1	+4.7	Sept. 2–9.
7	3 to 10	A	8–15	21.6	+5.5	+1.4	+7.9	Aug. 23–Sept. 2.
8	3 to 15	B	0–5.5	18.4	+2.0	−3.5	+7.9	Sept. 27–Oct. 6.

[1] Plus and minus signs indicate moisture contents above and below optimum.

tributor and two disk harrows. When moving at the speed of 250 feet per minute, the water was applied at the rate of 0.5 gallon per square yard. Two trips with a disk harrow operating at 200 feet per minute was generally required for mixing the water and soil. While these operations sometimes delayed the roller, they were not responsible for delays in hauling or spreading.

REQUIRED COMPACTION READILY OBTAINED

The construction records summarized in tables 11 and 12 show that the required compaction was obtained with all types of rollers.

The average density and average number of trips by rollers of each type required to obtain compaction are shown in table 16.

TABLE 16.—*The average density and number of trips needed for compaction*

Roller type	Number trips		Maximum wet density in Indiana	Maximum dry density in Ohio
	Indiana	Ohio		
			Percent	*Percent*
Sheepsfoot	7.6	7.6	97.2	98.6
3-wheel	2.1	3.4	99.5	102.9
Penumatic tire	3.1	5.1	98.1	102.9

Indiana Project.—The three-wheel and pneumatic tire rollers gave equal compaction with the same effort regardless of whether the loose thickness of the lift was 6, 9, or 12 inches. The data disclose also that variations in average moisture content of the soil of as much as 5.5 above the optimum, and more than 7 in a few instances, did not increase the amount of rolling necessary to obtain the specified density.

Extreme cases of 18 trips with the sheepsfoot roller, 5 trips with the three-wheel roller, and 12 trips with the pneumatic tire roller were recorded. This occurred in only one instance with each type of roller and had little effect on the average values. As a matter of fact, the specified density was obtained with 6 or less trips on 69 percent of the lifts compacted with the sheepsfoot roller, with 11 to 13 trips on 24 percent of the lifts, and with 8 or 9 trips on 5 percent of the lifts. Likewise, 2 or less trips were required on 88 percent of the lifts compacted with the three-wheel roller, and 3 or less trips on 88 percent of the lifts compacted with the pneumatic tire roller.

Several tests were made on section 6 to determine if there was any difference in density between the upper and lower halves of a 12-inch lift compacted by a three-wheel, 10-ton roller. The results of these tests, given in table 17, show that satisfactory compaction was produced throughout the entire thickness of the lift and that the lower half is likely to have a density equal to or greater than the upper half. The moisture content of the soils in this section ranged from 0.1 below to 4.7 above the optimum, and averaged 2.7 above the optimum. It has been observed that these relatively high moisture contents facilitate compaction of soils such as were found on this project.

Ohio Project.—With the three-wheel and pneumatic tire rollers slightly more rolling was necessary on the 9-inch than on the 6-inch lifts in order to obtain equal compaction.

Extreme cases of 20 trips with the sheepsfoot roller and 8 trips with the three-wheel and pneumatic tire rollers were recorded. This occurred on only one lift in the case of the sheepsfoot roller and on three lifts with each of the other rollers. The construction records show that the specified density was obtained with 8 or less trips on 77 percent of the lifts compacted with the sheepsfoot roller, 4 or less trips on 74 percent of the lifts compacted with the three-wheel roller, and 6 or less trips on 93 percent of the lifts compacted with the pneumatic tire roller.

TABLE 17.—*Results of density determinations on upper and lower halves of 12-inch lift compacted with 3-wheel 10-ton roller in Indiana*

Lift No.	Station	Number trips with roller	Maximum wet density	
			Upper half	Lower half
			Percent	*Percent*
7	271+02	2	105.9	101.0
8	270+32	2	105.1	111.2
8	270+35	2	102.3	109.8
9	271+02	2	100.4	106.6

The average densities of the compacted fills were very satisfactory. Although the minimum densities in some cases were considerably below the percentage specified, it will be seen that all lifts, with the exception of those in section 8, were compacted to a density of more than 90 percent of the maximum as indicated by the compaction test. Only three lifts in section 8 fell below this mark. These three lifts had densities equal to 83, 88.8, and 89.5 percent of the maximum.

ROLLER PRODUCTION DISCUSSED

Despite the fact that the required compaction was obtained by fewer trips of the three-wheel roller than by other types of rollers higher production in cubic yards compacted per hour was obtained with the pneumatic tire and sheepsfoot rollers on the Indiana project where the fill material was placed in 6-inch lifts and on the Ohio project for both 6- and 9-inch lifts. Differences in width and speed of roller account for this.

For example, sheepsfoot roller type A covered a strip 106 inches wide as compared with 60 inches for the pneumatic tire roller, while for the three-wheel roller the aggregate width of the rear rolls was 46 inches on the Indiana project and 40 inches on the Ohio project. At equal speeds the sheepsfoot roller could roll a given area once in 57 percent of the time required by the pneumatic tire roller, in 43 percent of the time required by the three-wheel roller in Indiana and in 38 percent of the time required by the three-wheel roller in Ohio.

Stated in another manner at equal operating speeds the sheepsfoot roller could roll an area approximately 1.8 times as fast as the pneumatic tire roller, 2.3 times as fast as the three-wheel roller in Indiana, and 2.6 times as fast as the three-wheel roller in Ohio.

Speed of Rollers in Indiana Project.—The average speed of the sheepsfoot roller of 256 feet per minute on section 1 was the maximum obtainable with the 40-horsepower tractor used to pull the roller. This tractor did not have the power necessary to operate at a higher speed.

Speed of the pneumatic tire roller was influenced by safety considerations. The roller had a tendency to tip over, so it was necessary to operate at slow speed along the edges and sides of the fill. Speed was increased on the central portion of the fill. This difficulty

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was not encountered with the sheepsfoot roller which could readily adapt itself to uneven and irregular surfaces.

In accordance with the specification requirements, the three-wheel roller was operated in extreme low gear at approximately 3 feet per second. On one lift of section 2 the speed was increased to about 6 feet per second. The density determination showed that the compaction was undiminished at the higher speed and that an increase in roller capacity corresponding to the increase in speed could be obtained.

Relative Production in Indiana Project.—Table 13 shows that the pneumatic tire roller compacted a cubic yard of soil in 6-inch lifts in less time than the other rollers. The greatest number of trips of actual rolling was recorded for the three-wheel roller on section 2. The maximum production of 361 cubic yards per hour was obtained with the pneumatic tire roller on section 8 as compared with 244 for the three-wheel roller on section 2, and 341 and 255 for the sheepsfoot roller on sections 1 and 3, respectively.

With respect to the production of the sheepsfoot roller on sections 1 and 3, the greater production was obtained on section 1 even though the average number of trips required to obtain the specified density was 5 percent higher. The increase in production of 33 percent is attributed entirely to the increase of 38 percent in the roller speed.

On the sections where the material was placed in lifts having loose thicknesses of 9 and 12 inches, greater production was obtained with the three-wheel roller than with the pneumatic tire roller. The sheepsfoot roller was used only on the 6-inch lifts. A very definite increase in production was accomplished by increasing the lift thickness. The influence of lift thickness on the roller production is shown in figure 26.

Ohio Project.—Table 14 shows that on the sections where soil was placed in 6-inch lifts 12.9 seconds was required to compact a cubic yard of soil with the sheepsfoot roller on section 4 and 19.9 seconds on section 1. These figures compare with 16.4 seconds for the pneumatic tire roller on section 3 and 19.9 seconds for the three-wheel roller on section 2. The least amount of time necessary to compact 1 cubic yard on the sections constructed in 9-inch lifts was 13 seconds with the pneumatic tire roller.

Maximum production of 279 cubic yards compacted per hour was obtained with the sheepsfoot roller on section 4. On section 1, however, the sheepsfoot roller was able to compact only 181 cubic yards per hour. The greater production on section 4 indicates that the higher moisture content of the soil was significant in facilitating compaction. The average variation from the optimum moisture content on section 4 was −1 as compared with −2 on section 1. The percentages of the total number of lifts compacted at moisture contents below the optimum on sections 1 and 4 were, respectively, 94 and 74 percent.

The average number of cubic yards compacted per hour for the different rollers are summarized as follows:

Sheepsfoot roller:	
6-inch lift	245 cu. yd.
9-inch lift	255 cu. yd.
All sections	247 cu. yd.
Pneumatic tire roller:	
6-inch lift	220 cu. yd.
9-inch lift	277 cu. yd.
All sections	227 cu. yd.
Three-wheel roller:	
6-inch lift	181 cu. yd.
9-inch lift	247 cu. yd.
All sections	216 cu. yd.

These data disclose maximum production with the sheepsfoot roller on the 6-inch lifts and with the pneumatic tire roller on the 9-inch lifts. Although the increase in production obtained with the sheepsfoot roller as a result of increasing the lift thickness is very small, the production of the pneumatic tire and three-wheel rollers was increased materially. The influence of lift thickness on roller production is shown in figure 27.

Variations in production and compaction corresponding to different operating speeds of the rollers, as summarized from the daily records of the production studies, are given in table 18. Although it was im-

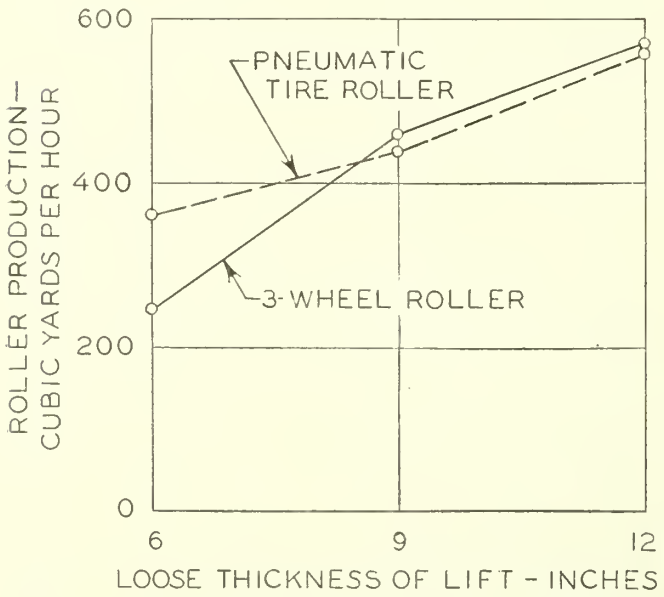


FIGURE 26.—RELATION BETWEEN LIFT THICKNESS AND ROLLER PRODUCTION IN GIBSON COUNTY, INDIANA.

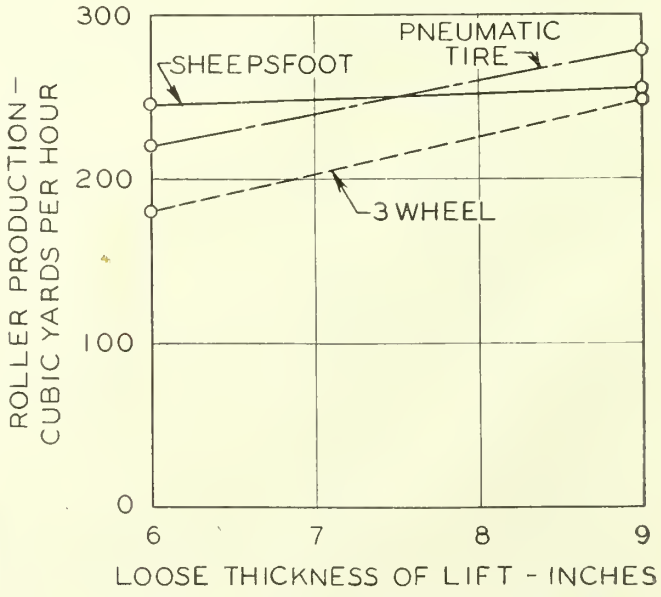


FIGURE 27.—RELATION BETWEEN LIFT THICKNESS AND ROLLER PRODUCTION IN DELAWARE COUNTY, OHIO.

possible to maintain a constant rate of speed under the various construction conditions, it is believed that the speeds indicated represent the average speed during the period of observation. The data show a wide variation in volume of fill compacted per hour for the different roller speeds. There is, however, a definite indication that greater fill production was obtained at the higher speeds without any sacrifice in the quality of the compaction.

PERFORMANCE OF SHEEPSFOOT ROLLERS COMPARED WITH THAT OF OTHER ROLLERS

Ohio Project.—Of the 100 lifts on which the sheepsfoot rollers were used, 87 were compacted with type A roller and 13 with type B roller. The average densities obtained with each roller and the average number of trips required to obtain this density were as follows: Type A roller with 7.8 trips produced 98.7 percent of maximum dry density and type B roller with 6.8 trips produced 97.7 percent.

Type B roller was used only on eight lifts of section 4, and five lifts of section 6. The remaining lifts on these sections were compacted with type A roller. On section 4 both rollers were loaded to capacity and an attempt was made to obtain the specified 100 percent compaction.

On section 6, in addition to using both rollers with the drums full of water, roller type B was used with the drums about half full of water. While in this condition all lifts were compacted by six trips and the soil densities determined. The densities obtained were considered satisfactory.

The results obtained on sections 4 and 6 are summarized in table 19. These data indicate that slightly

TABLE 18.—*Effect of roller speed on production and compaction in Delaware County, Ohio*

Section No.	Type roller	Roller speed	Compaction [1]	Volume compacted per hour of rolling	Average roller trips to obtain compaction
		Feet per minute	*Percent*	*Cubic yards*	
1	Sheepsfoot	250	100.1	172	8
		300	96.8	238	8
		300	91.8	172	10
		300	103.0	172	12
2	3-wheel	100	108.2	75	3
		150	104.6	232	3
		200	110.1	264	3
		200	105.1	138	4
		300	104.3	294	3
3	Pneumatic tire	200	98.7	166	2
		200	105.6	164	5
		300	92.5	250	3
		300	101.7	223	4
		300	105.8	262	5
		300	104.0	244	6
4	Sheepsfoot	200	100.1	222	6
		200	106.4	158	7
		230	100.3	257	6
		240	102.2	332	6
		240	101.3	176	9
		250	100.7	300	6
		270	101.6	408	6
		300	100.8	441	5
		330	96.0	380	6
5	3-wheel	200	106.7	191	2
		240	104.0	236	3
		250	99.7	244	2
		300	107.7	269	4
		300	109.3	174	5
		330	99.2	290	5
		340	102.9	274	5
		400	109.8	317	5
6	Sheepsfoot	200	100.0	194	6
		360	95.9	285	6
7	Pneumatic tire	300	100.0	319	6
		300	101.2	306	7
		400	102.2	258	8

[1] Compaction in percentage of maximum dry density.

FIGURE 28.—COMPACTED FILL IN OHIO AFTER RAIN. THE SURFACE IS SLICK AND SLUSHY BUT NOT RUTTED.

TABLE 19.—*Summary of roller trips and densities on sections 4 and 6 in Delaware County, Ohio*

Roller type	Quantity of water in drum	Number lifts	Number trips	Maximum dry density [1]
				Percent
Section 4:				
A	Full [2]	20	5.9	101.9
B	Full [3]	8	7.3	99.1
Section 6:				
A	Full [2]	8	6	97.9
B	Partly filled [4]	3	6	94.7
B	Full [3]	2	6	95.6

[1] Average values.
[2] Ground pressure of 223 pounds per square inch.
[3] Ground pressure of 290 pounds per square inch.
[4] Ground pressure of 232 pounds per square inch.

better results were obtained with roller type A than with roller type B, and that roller type B was more effective with the heavier load. Since there were variations in soil and moisture content at the time of compaction and the data are limited, it seems that the differences in compaction are not great enough to be significant.

OPERATION OF SHEEPSFOOT ROLLERS IN INDIANA IMPROVED BY LOAD ADJUSTMENT

To meet the requirements of the specifications relating to the ground pressure under each tamping foot of sheepsfoot roller type A, it was necessary to fill the drums with water. There was a tendency at times for the tamping feet to pick up the soil and tear material loose rather than compact it. This was overcome by drawing out part of the water and reducing the ground pressure under each foot from 209 to 164 pounds per square inch. The required compaction was readily obtained in this manner.

Sheepsfoot roller type B was used only on 6 of the 20 lifts in section 1. It was first tried with the drums full of water. Under this load the feet dug up the soil and satisfactory compaction could not be obtained. It was then used with the drums half filled with water and gave good results. The most efficient operation was obtained when the drums were empty. In this condition the ground pressure under each tamping foot was approximately equal to that of sheepsfoot roller type A partially filled with water.

FIGURE 29.—EROSION IN INDIANA (A) OF THE SHEET AND GULLY TYPE IN BORROW PIT A, (B) CHANNELS DRAIN INTO OUTLET IN BORROW PIT A, (C) IN FILL SLOPE BEFORE GOOD COVER OF LESPEDEZA WAS ESTABLISHED, AND (D) TYPICAL CONDITION OF SHOULDERS AND SLOPES ON ENTIRE PROJECT.

COMPACTION PRODUCED BY BULLDOZERS AND TRACTORS IN INDIANA COMPARED

The compaction produced by construction equipment other than rollers was investigated on lifts 5 and 6 of section 7. Density tests were made in the path of the bulldozer and tractors spreading the material deposited by trucks. These tests were in addition to those performed after compacting with the pneumatic tire roller, which was used on portions of lifts 5 and 6, as well as on the other lifts comprising the fill section. The results of these tests are presented in table 20.

The normal movements of the 17-ton bulldozer in spreading the soil on lift 5 produced a density of 94.4 percent of the maximum wet density. Two passages of the 11-ton tractor increased the density to 97.5 percent. This compares with 101.1 percent obtained with three trips of the pneumatic tire roller.

On lift 6 the density obtained with the 17-ton bulldozer was 93.3 percent of the maximum wet density. In one location, two passages of the 11-ton tractor did not increase the density, while in another instance the density was increased to 96.2 percent. This difference may be accounted for by the fact that in the former case the lower 2 inches of the lift was loose and uncompacted.

Two trips of the pneumatic tire roller over the bulldozed material did not serve to increase the density on lift 6. However, an average density of 97.3 percent was obtained on this fill section with the pneumatic tire roller, and in no case were more than three trips necessary to obtain the required density of 95 percent. The moisture contents of the fill material in this section ranged from 1.4 to 7.9 above the optimum and averaged 5.5 above the optimum.

COMPACTION PREVENTED SOFTENING FROM RAINS BUT DID NOT PREVENT EROSION

Removal or mixing with drier soil of any layer of the compacted embankment, which became softened

TABLE 20.—*Compaction produced by bulldozer, tractors, and pneumatic tire roller on lifts 5 and 6 of section 7 in Gibson County, Indiana*

Type of equipment	Number trips	Lift No.	Maximum wet density
			Percent
17-ton bulldozer	Spreading [1]	5	94.4
11-ton tractor	2 [2]	5	97.5
Pneumatic tire roller	3 [3]	5	101.1
17-ton bulldozer	Spreading [1]	6	93.3
11-ton tractor	2 [2]	6	[4] 93.1
Pneumatic tire roller	2 [3]	6	93.4
11-ton tractor	2 [2]	6	96.2

[1] Normal movement of bulldozer spreading material. Number of trips indefinite.
[2] Tractor moved forward and backward in same path after bulldozer had completed spreading.
[3] Roller used on portions of lifts after bulldozer had completed spreading.
[4] Loose, uncompacted material found in lower 2 inches of lift.

by rains was a requirement of the specifications for the Ohio work. Although heavy rains occurred during the construction of the test sections, they did not cause softening of the compacted lifts. The surface of the fill became slick and a small amount of slush was formed under the action of traffic as shown in figure 28 but there was no detrimental rutting. Only removal of the slushy material by blading was needed to accommodate traffic over the fills without any inconvenience.

The contractor was able to continue grading operations within a few hours after it stopped raining. Unsatisfactory condition developed only on the lifts compacted with the sheepsfoot roller. The small depressions left by the feet of the roller held water and became quite soft, necessitating considerable drying before additional material could be placed. This condition was overcome by smoothing out the surface with the three-wheel or pneumatic tire roller just as the rain was commencing.

As illustrated in figure 29, *A* and *B*, the soils used in the construction of the fills in Indiana were susceptible to severe sheet and gully erosion when not protected by a good cover of vegetation. A great many washouts and gullies were observed on the shoulders and slopes of all the sections soon after the fills were completed. Following the construction of the concrete pavement, the shoulders and slopes were repaired and planted with Lespedeza. Some erosion occurred (fig. 29, *C* and *D*) soon after planting but this has been reduced to negligible amounts since a satisfactory cover has been established.

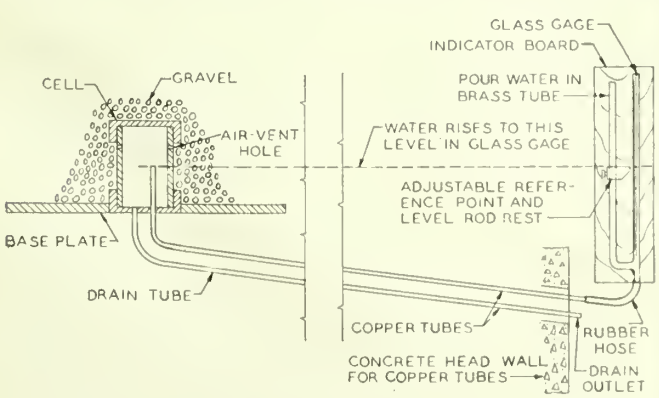
Embankment slopes in Ohio were left unprotected for the purpose of observing the resistance to erosion of the sections compacted by different types of rollers.

On neither of the projects was there a significant difference in condition of the various fill sections indicating that the edges of the fills were equally well compacted by all the rollers insofar as resistance to erosion was concerned.

SETTLEMENT CELLS INSTALLED

For the purpose of measuring the subsidence of the fills into the undersoil use was made of settlement cells. Total settlement of the fills including subsidence was determined from measurements made on reference stakes, plugs, and marked locations on the pavement.

The cells used to measure the subsidence are similar to the Ames settlement cell described in Bulletin 112 of the Iowa Engineering Experiment Station, published in 1933. The cell, and its accessory parts, illustrated in figure 30, operates on the principle that the level of liquids in two vessels connected by a tube will be the same, provided the tube is large enough to allow the liquid to flow freely.



URE 30.—Diagram Showing Operation of Settlement Cell.

Figure 31.—Placing Settlement Cell in Ohio.

The cell (fig. 31) consists of a small chamber welded to a steel plate 12 inches square. It can be placed at any point in the embankment. Within the chamber are two orifices at different levels. The upper orifice, consisting of a tube screwed to the base of the cell, is coupled to a copper tube extending out through the fill to a convenient place where it is attached by means of a rubber hose to a glass gage tube mounted on an indicator board (fig. 30) which may be moved from one location to another. The lower orifice, flush with the base of the cell, is connected with another copper tube and serves merely as an overflow outlet for the system.

When taking readings to determine the changes in elevation of the cell, the indicator board with the attached glass gage and brass tubes is set in a vertical position near the toe of the fill slope, as illustrated in figure 32. By means of a rubber hose, the glass gage is connected to the copper tube leading to the upper orifice in the cell chamber. The other tube is left open to serve as a drain. Water is poured into the brass tube until it overflows the upper orifice and flows out from the outlet pipe. Water is added until the system is entirely free of air as indicated by a uniformly continuous flow from the drain tube. The level of the water in the glass gage is then the same as the upper orifice in the cell chamber. The adjustable reference point is moved vertically on the brass tube to coincide with the level of the water in the glass gage.

The adjustable reference point also served as a rest for the level rod when determining the elevation of the water in the glass gage.

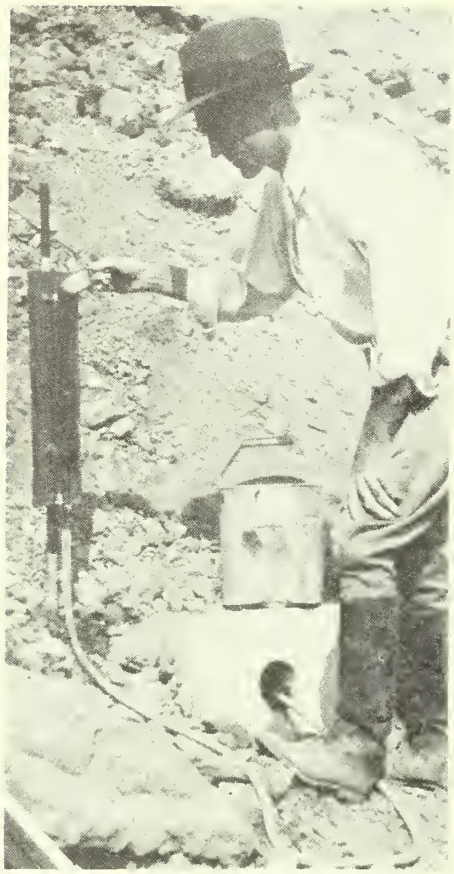


FIGURE 32.—POURING WATER INTO GAGE ON OHIO PROJECT.

Air-vent holes drilled near the top of the cell serve to break suction or siphonic action when water is poured into the tube quite rapidly.

Cells were installed either on the original ground surface under the fill or on top of the foundation layer on the Indiana project and on top of the old roadway fill on the Ohio project. All measurements were referenced to frostproof bench marks situated at convenient locations outside of the fill areas.

The locations of the cells and bench marks are shown on figures 3 and 4. The design details of the frostproof bench marks and their method of installation are illustrated in figure 33.

The arrangement of the settlement cells and accessories on the Indiana project is illustrated in figure 34, *A*, and on the Ohio project in figure 34, *B*. A trench about 12 inches wide was excavated from the center line where the cell was to be placed perpendicular to the center line to a point just outside the fill slope. The bottom of the trench was sloped toward the outer end to give a drop of about 18 inches.

The two ¼-inch copper tubes, which were of sufficient length to reach beyond the fill slope, were coupled to the cell and placed along the bottom of the trench.

On the Indiana project a thin layer of gravel was spread over an area slightly greater than would be covered by the base plate of the cell. The cell was seated firmly on the gravel or on the ground (in Ohio), and the tubing was straightened to eliminate kinks. Gravel (about 1 cu. ft. in Indiana) or crushed stone (about 2 cu. ft. in Ohio) was placed over the cell to provide air space around the air-vent holes, and the

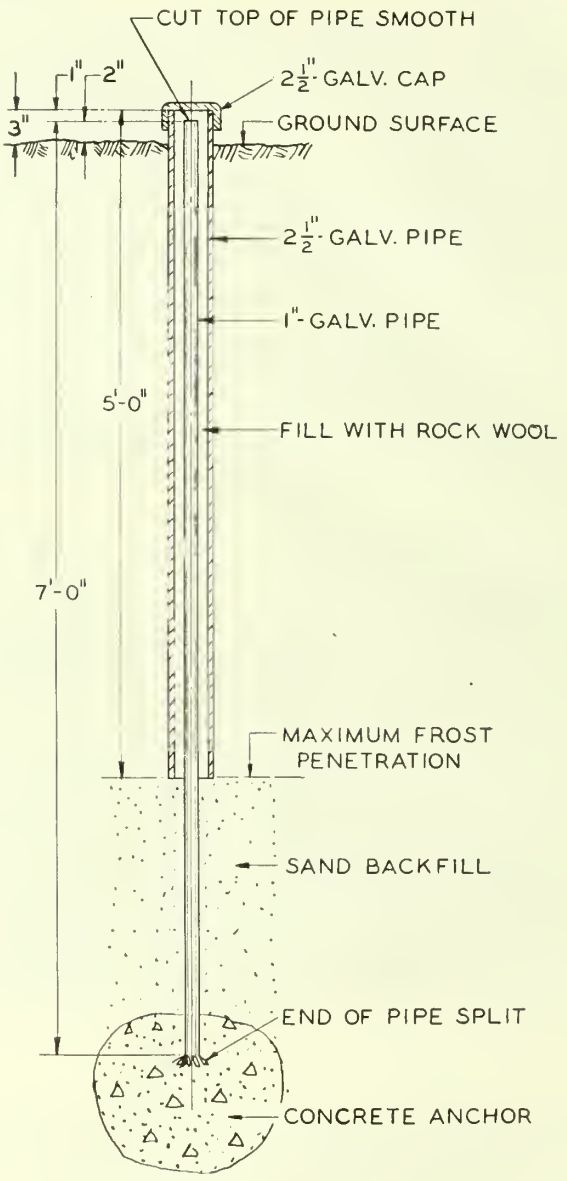


FIGURE 33.—BENCH MARK.

trench containing the copper tubes was backfilled with soil and tamped by hand.

A mound of soil about 15 inches high was placed over the cell and tubes to protect them from damage by construction equipment. Soil was delivered by trucks and deposited on each side of the tubes. The soil was spread by a bulldozer, care being taken not to pass over the cell until the mound was completed. The protective mound of soil was from 15 to 20 feet in width.

The copper tubes were protected where they emerged from the fill by the concrete head wall illustrated in figure 32.

REFERENCE STAKES, PLUGS, AND MARKS PLACED ON TOP OF FILLS

In Indiana the settlement occurring in the fill prior to the construction of the concrete pavement was determined by taking elevations on reference stakes set in the top of the completed embankment directly over the points where the settlement cells were placed. The stakes were cut in lengths of about 16 inches from 4-inch green saplings and were painted with motor oil

to retard cracking. A 2-inch boat spike was driven in the top of each stake.

A hole, 8 inches in diameter and 12 inches deep, was dug with a post-hole digger and the stake, after being again painted with oil, was placed on undisturbed soil in the hole. A 2-inch layer of loose soil was placed in the hole around the stake and oil was poured on the soil. After the oil had penetrated the soil, the layer was tamped, and another layer was placed in the same manner.

Oil-saturated rock wool was used instead of oil-treated soil for the stakes located over cells 12, 13, and 14. The rock wool was mixed with oil and tamped in place in layers having a compacted thickness of 1 inch. The hole was filled with rock wool to within 4 inches of the top, and filling was completed with oil-treated soil.

All these precautions were taken to eliminate the effects of frost action. In addition, a covering of straw was placed over the stakes during the winter months. Three 2-inch-square guard stakes were placed around each settlement stake to protect it from damage by cars or equipment.

After the concrete pavement was constructed during August 1939, a brass plug was installed at the location of each cell, at the quarter points of the pavement. The details of the plug and its installation are shown in figure 35.

In Ohio settlements occurring in the different fill sections were measured on the east curb of the pavement at points located at the same stations as the settlement cells. These points on the curb were marked by a 2-inch cross chiseled in the concrete and painted with red enamel. The marks were cut in the curb during the period from October 24 to 28, 1938, and the elevations determined at that time.

SETTLEMENT AND SUBSIDENCE CAREFULLY OBSERVED

Indiana Project.—Readings on the settlement cells were taken two or three times a week while the fill was being placed; once every 10 or 15 days from the completion of the fill section until the completion of the entire job on October 12, 1938; and subsequently on November 10 to 14, 1938, May 22 to 23, 1939, April 26 to 28, 1940, June 10 to 12, 1940, December 10 to 11, 1940, and November 17 to 19, 1941.

Readings on the reference stakes on top of the fill were taken when the readings on the settlement cells were taken and, in addition, on March 20 and May 8, 1939. The last measurements were made on the stakes on May 22 and 23, 1939, just prior to the construction of the concrete pavement. Readings on the reference plugs were taken on June 11 and 12, 1940, December 10 and 11, 1940, and November 17 and 19, 1941. The results of the measurements on cells, stakes, and plugs are summarized in table 21.

The total settlement of the fill sections as measured on the brass plugs installed in the pavement ranged from zero to 0.03 foot with an average of 0.01 foot. These movements were recorded during the period from June 1940 to November 1941.

The maximum settlement as measured on the wooden stakes from completion of the fill to the construction of the pavement was 0.09 foot at the location of cell No. 8 in section 4. The average for sections 2 to 8 was 0.05 foot. An average upward movement of 0.015 foot was recorded in section 1.

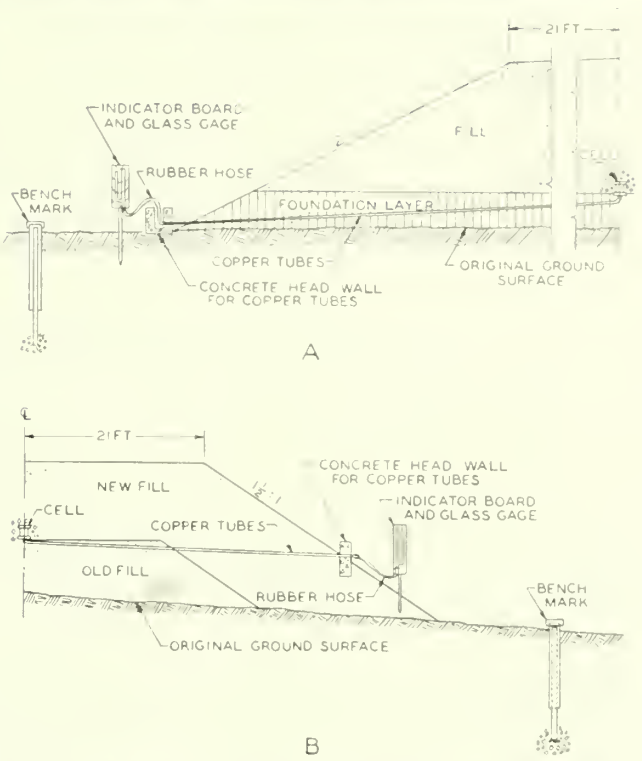


FIGURE 34.—INSTALLATION OF SETTLEMENT CELL AND ACCESSORIES IN INDIANA (A) AND IN DELAWARE COUNTY, OHIO (B).

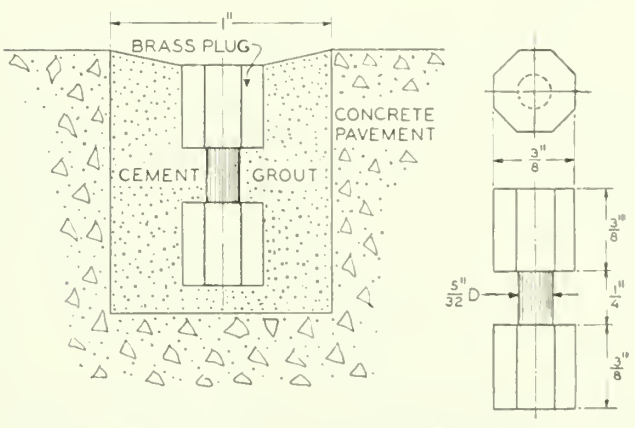


FIGURE 35.—DETAILS OF BRASS PLUGS SET IN PAVEMENT IN GIBSON COUNTY, INDIANA.

Ohio Project.—Settlement cell readings were taken each day while the fill was being placed, once a week during the period from the date each section was completed until the date of completion of the entire project on October 24, 1938, and subsequently in February, April, July, and November 1939, July and December 1940, and January 1942. After October 28, 1938, measurements at the marked locations on the pavement curb were taken on the same dates as the readings on the settlement cells. The results of the readings on the settlement cells and on the east curb are plotted in figure 36 and summarized in table 22.

Although the concrete base and curb were constructed almost immediately after completion of the embankment (2 to 28 days), the total settlement taking place within the body of the fill as measured on the east curb did not exceed 0.08 foot. The curb in sections 1 and 6

TABLE 21.—*Summary of measurements of vertical movements of settlement cells under fill, wooden stakes in top of fill, and brass plugs in pavement in Gibson County, Indiana*

Section No.	Cell No.	Movement of settlement cell			Total movement of wooden stakes	Movement of cells during period of total movement of wooden stakes	Total movement of brass plugs [1]	Movement of cells after brass plugs were installed [2]
		During fill construction	Total to May 1939	Total to November 1941				
		Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1	1	[3]−0.15	−0.22		[4]+0.01	−0.07		
	2	−.12	−.08	0.15	+.02	+.04	0.00	−0.03
2	3–A	+.26	−.70	.60	−.08	−.96	−.02	−.42
3	5	−.70	−1.29	−.43	−.06	−.66	−.01	.00
4	7	−.12	−.12	−.05	−.05	.00	.00	+.24
	8	−.09	−.37	−.52	−.09	−.28	−.01	+.38
5	10	−.16	−.23	−.24	−.06	−.07	−.01	+.24
6	11	−.32	−.45	−.29	−.05	−.13	−.02	−.14
	12	−.25	−.59	−.23	−.06	−.34	.00	+.03
7	13	−.23	−.26	−.22	−.05	−.03	.00	−.01
	14	−.18	−.23	−.17	−.05	−.05	.00	+.02
8	15	−.05	−.01	−.08	−.01	+.04	−.03	−.10
	16	−.10	−.08	−.06	.00	+.02	−.02	+.01

[1] Values are averages of measurements made on two plugs installed at quarter points on each side of center line for the period from June 1940 to November 1941.
[2] Differences in elevation from June 1940 to November 1941.
[3] − denotes downward movement.
[4] + denotes upward movement.

started to move upward soon after construction and at the last readings had not returned to its original elevation. However, the maximum uplift (see fig. 36) never exceeded 0.04 foot at any time during the period of observations. The average settlement on six of the nine sections was 0.025 foot while an average upward movement of 0.017 foot was recorded on the other three sections.

BEHAVIOR OF SETTLEMENT CELLS DESCRIBED

Indiana Project.—Cell Nos. 3, 4, and 6, placed directly on the natural ground surface in sections 2 and 3 which were the first constructed, were soon put out of operation by action of the grading equipment in placing the foundation layer. Cell No. 1 in section 1 and cell No. 16 in section 8 were installed on the natural ground surface after a prolonged dry period. The remaining cells were placed on top of the foundation layer.

Cell No. 1 functioned satisfactorily until May 22, 1939, after which it could not be made to operate and all readings were suspended at this location.

Cell Nos. 2, 7, 8, 10, 13, 14, and 15 were functioning in a satisfactory manner when the readings were taken in November 1941. A uniform flow of water was always obtained through the cell and the level of the water in the glass gage remained constant. When water was run through the cell a second time, the first reading was readily checked. The entire operation required about 15 minutes.

The readings taken in November 1941 disclosed that cell Nos. 2, 7, and 8 were becoming sluggish in their operation.

Cell No. 16 did not respond readily when first installed. A considerable amount of water was added before any flowed from the outlet. After excavating some of the fill material near the toe of the slope, it was found that the tubes were curved upwards. As soon as they were straightened, the flow became uniform and the normal amount of water was required.

Cell No. 3–A behaved very sluggishly. A steady flow of water was obtained through the cell, but the level in the gage fluctuated over a range of as much as 5 inches when the water was cut off. The fluctuation in water level gradually decreased to a range of one-half inch in 15 minutes. When this fluctuation was reduced to about one-fourth inch, the indicator was set at the average level and a check was made. Several determinations were sometimes necessary before a reading was checked.

More time was required to determine the elevation of cell No. 5 than for any of the others. When apparently at equilibrium, the water level in the gage dropped suddenly and then slowly moved upward, coming to rest after about 45 minutes.

The action of cell Nos. 11 and 12 was quite similar. Each required the ordinary amount of water to obtain a uniform flow through the cell. When the water supply was shut off, the level fluctuated. The fluctuation was very irregular and did not gradually decrease as in the case of cell No. 3–A. The level would drop quickly and rise slowly, or vice versa, and the change in level might be 6 inches or a very slight amount. A constant level might be reached in a few minutes or it might take nearly an hour.

Ohio Project.—Erratic readings were attributed in part to adjustments in the seating of the cells resulting from the operation of heavy equipment in placing and compacting the fill material. Evidence of such action is the large amount of settlement occurring, for example, in section 2, on the first day following the installation of the cell. There seems to be no reason for this behavior except that the foundation for the cell was disturbed when the cell was installed.

Also, it is very likely that kinks had developed in the soft copper tubes extending through the fill. When

TABLE 22.—*Summary of readings taken on settlement cells and on east curb in Delaware County, Ohio*

Section No.	Construction period in 1938				Time from completion of fill to construction of base and curb	Initial readings in 1938				Vertical movement of cell [1]				Total movement of east curb to January 1942 [1]
	Fill		Base and curb			Cell		Curb		During construction of fill	During construction of fill and pavement	Total to January 1942	From date of initial reading on curb to January 1942	
	Started	Finished	Started	Finished		Date	Elevation	Date	Elevation					
					Days		*Feet*		*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1	9–19	9–25	10–18	10–19	24	9–20	907.88	10–24	914.73	−0.04	−0.09	−0 10	−0.01	+0.01
2	9–25	10–5	10–19	10–20	15	9–27	905.18	10–24	912.83	−.24	−.29	−.29	.00	−.01
3	9–24	10–5	10–21	10–22	17	9–28	898.51	10–28	908.83	−.01	−.02	−.09	−.07	−.03
4	10–4	10–19	10–24	10–25	6	10–5	913.47	10–28	922.99	−.05	−.07	−.08	−.01	−.08
5	10–14	10–24	10–26	10–26	2	10–16	923.09	10–28	934.27	−.08	−.08	−.13	−.05	−.01
6	10–11	10–15	10–27	10–27	12	10–12	917.88	10–28	923.07	−.05	−.05	−.07	−.02	+.03
7	10–7	10–10	10–15	10–15	5	10–8	922.71	10–24	927.13	−.05	−.07	−.04	+.03	−.01
8	8–31	9–19	10–10	10–10	21	9–5	918.47	10–24	928.79	−.18	−.24	−.27	−.03	−.01
9	8–26	9–12	10–10	10–10	28	8–30	916.95	10–24	927.02	−.12	−.20	−.22	−.02	+.01

[1] Figures show difference in elevation between dates of initial and final readings for period covered regardless of fluctuations during that period. (See figure 36.)

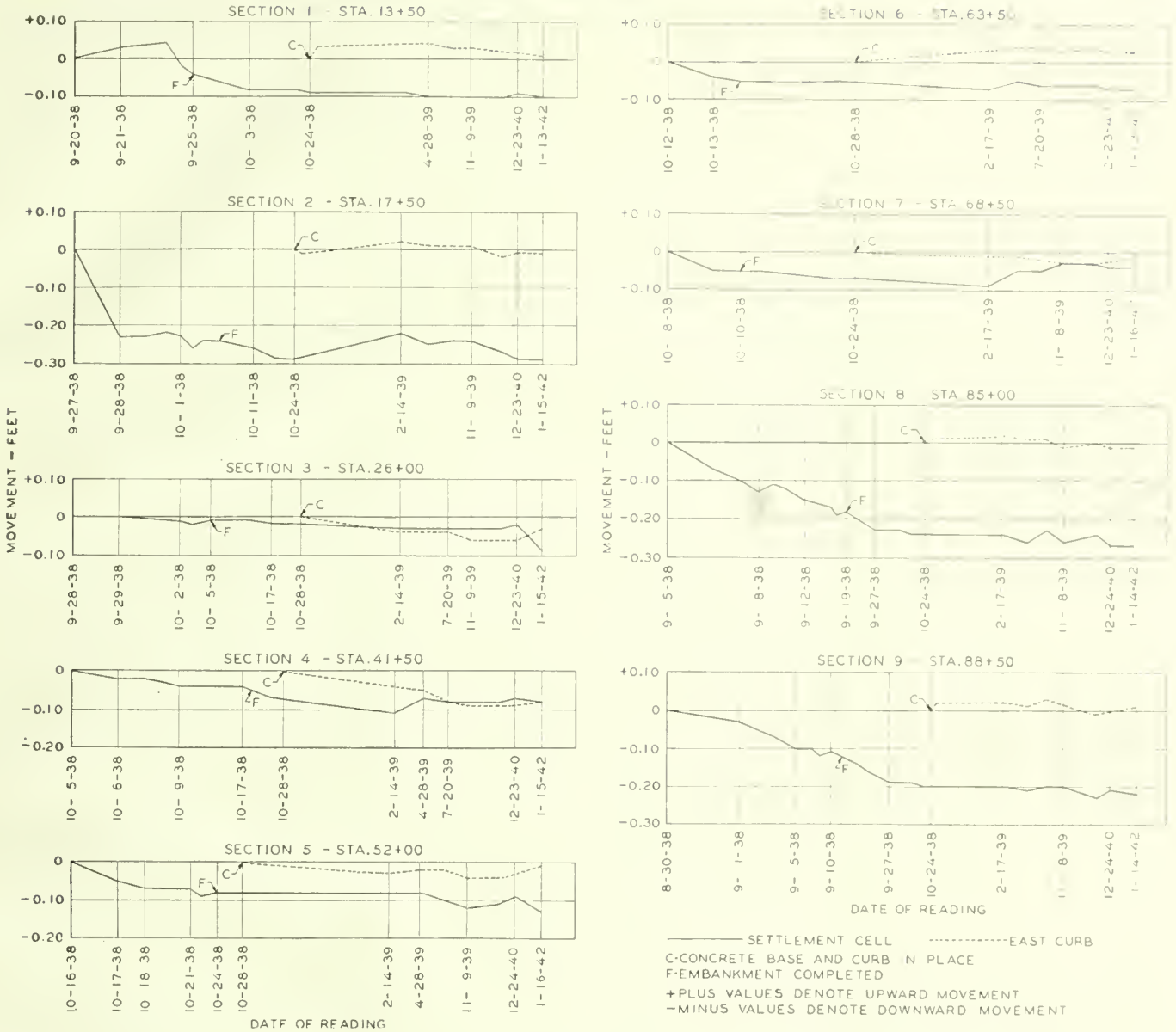

SECTION 1 - STA. 13+50
SECTION 2 - STA. 17+50
SECTION 3 - STA. 26+00
SECTION 4 - STA. 41+50
SECTION 5 - STA. 52+00
SECTION 6 - STA. 63+50
SECTION 7 - STA. 68+50
SECTION 8 - STA. 85+00
SECTION 9 - STA. 88+50
MOVEMENT - FEET
DATE OF READING
SETTLEMENT CELL
EAST CURB
C-CONCRETE BASE AND CURB IN PLACE
F-EMBANKMENT COMPLETED
+PLUS VALUES DENOTE UPWARD MOVEMENT
-MINUS VALUES DENOTE DOWNWARD MOVEMENT

FIGURE 36.—RESULTS OF MEASUREMENTS ON SETTLEMENT CELLS AND EAST CURB FILL CONSTRUCTION EXPERIMENT, DELAWARE COUNTY, OHIO.

testing the operation of a cell prior to installation under the fill, it was observed that kinks in the line interfered with the free flow of water through the system and made it difficult to check readings.

Both Projects.—The purpose of the measurements of settlement on the top of the fill or pavements was to indicate the relative stability of the various fill sections compacted by different rollers and with various thicknesses of layers.

It had been assumed that differences between the total settlement as measured on the top of fill or pavement and the settlement as measured by the cell readings would disclose the amount of consolidation that had occurred within the fill. However, the data presented in tables 21 and 22 show that the subsidence indicated by the cells had no direct relation to the total settlements on either project. Upward movements of the cells were recorded at the same time that downward movements were measured on the stakes, plugs, and pavement curb, and vice versa.

Upward movements were recorded on the settlement gages at different times during the entire period of observation. Those occurring during construction may have been caused by adjustments in the seating of the cells or displacements of the soft undersoil resulting from the operation of the heavy equipment. However, the upward movements found a year or more after the completion of the embankment, cannot be attributed to foundation support but must be due to faulty operation of the cells or their accessory parts. As a result, the elevation measured on the gage outside of the fill did not accurately represent the elevation of the cell.

The behavior of cells is a strong indication that kinks developed in the soft copper tubes to the extent that they interfered with the free flow of water through the system.

Since the installation of the cells in these experiments, several improvements [6] have been made in the design of the system and in the method of seating the

[6] CONSOLIDATION OF EMBANKMENT AND FOUNDATION MATERIALS, PROGRESS REPORT OF SUBCOMMITTEE NO. 2 OF THE COMMITTEE OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION ON EARTH DAMS AND EMBANKMENTS. Proceedings of the American Society of Civil Engineers, vol. 66, No. 8, part 1, October 1940.

cell which tend to eliminate the difficulties described above.

CONCLUSIONS

The following conclusions seem justified in view of the fact that there have been neither indications of detrimental results from settlements nor indications of instability on any of the sections of either the Ohio or Indiana projects since the construction of the pavements.

Both Projects.—1. The compaction test may be utilized generally to control the construction of embankments regardless of the type of compacting equipment. It offers a practical means of determining when a layer of soil is satisfactorily compacted.

2. Data from compaction tests performed in the laboratory and from density tests made on the compacted fill offer a means of determining the moisture content and lift thickness at which satisfactory compaction may be obtained most economically with a given type of roller.

Indiana Project.—3. A density equal to 95 percent of the maximum as determined in the laboratory in accordance with method T 99–38 of the American Association of State Highway Officials is apparently sufficient to produce highway fills of satisfactory stability when the dimensions of the fills are similar to those on this project. The maximum height of fill in the various test sections ranged from 7 to 10 feet.

4. Soils similar to those comprising the fills on this project may be readily compacted to the desired density by any of the rollers used in this experiment if the moisture content of the soil is within the proper limits. With proper moisture content, equally good compaction was obtained throughout the entire thickness of a compacted layer regardless of whether the soil was compacted by the tamping action of the sheepsfoot roller, the kneading action of the pneumatic tire roller, or the compression produced by the three-wheel roller.

5. The moisture content of fills may be controlled within 1 of the optimum value when the soil is obtained from a borrow pit as was done on this project. Such rigid control, however, does not seem justified in light of the densities and stabilities obtained in this experiment with moisture contents ranging from 3.5 below to 7.9 above the optimum.

6. The high degree of stability attained at the wide range in moisture contents indicates that control of compaction on the basis of density alone will produce satisfactory results provided the soil contains sufficient moisture for the rollers to be effective and is not so wet as to interfere with the operation of the rollers. Definite limits above and below the optimum will depend entirely on the character of the soil. These limits may be determined by observing the performance of the rollers in conjunction with density tests.

7. The amount of fill material compacted per hour with a roller of a particular type depends on the type of soil, the moisture content of the soil, the thickness of the layer, and the speed of the roller. With soils and moisture contents typical of this project, it was found that when the loose thickness of the soil layer was increased from 6 to 12 inches and when the rollers were operated at their maximum speeds, a corresponding increase in roller capacity was obtained without any sacrifice in compaction.

Ohio Project.—8. Highway fills of satisfactory stability may be obtained when constructed in layers compacted to maximum density as determined on the same types of soils in accordance with the standard compaction test (Method T 99–38 of the American Association of State Highway Officials). Such embankments may be paved immediately following construction without danger of detrimental settlement.

9. Soils similar to those comprising the fills studied may be readily compacted to the desired density by any of the rollers used in this experiment when spread in layers having loose thicknesses of 6 to 9 inches. The soils ranged from silty clay loams to clays having physical properties of the A–4 to A–7 groups. They were of friable to hard consistency when dry but were plastic when wet.

10. A density equal to 95 percent of the maximum obtained in the compaction test is apparently satisfactory when the type of soils and the depth of the fills (5 to 11 feet over the old fill core and 7 to 21 feet on the widened portions) are similar to those on this project and when the moisture contents are reasonably close to the optimum.

11. Control of the percent of moisture within 1 of the optimum value is not practicable where the soil delivered to the fill is a mixture of materials varying in moisture-density characteristics. It is desirable that a greater permissible variation in moisture content from the optimum be established and this permissible variation should depend on the type of soil.

12. The use of moisture-density curves typical of the soils to be used and the selection of a particular curve in the manner described in this report facilitates the use of compaction test data under variable conditions, and is a practicable method of field control.

13. The construction of embankments may be controlled in accordance with the moisture contents and densities indicated by compaction test data without causing delay in construction operations.

14. The compaction of areas rolled at maximum roller speed is equal to that obtained at slower speeds. The higher speeds result in a corresponding increase in roller capacity.

PROSPECTING FOR GRAVEL DEPOSITS BY RESISTIVITY METHODS

BY THE DIVISION OF PHYSICAL RESEARCH AND TESTS, PUBLIC ROADS ADMINISTRATION

Reported by R. WOODWARD MOORE, Associate Civil Engineer

IN the spring of 1940 field tests were made by the Public Roads Administration in the vicinity of Beltsville, Md., to further investigate the usefulness of the earth resistivity method of test in locating gravel deposits.[1] Considerable quantities of gravel were needed in local road improvements within the Beltsville Research Center grounds. Shallow test pits had failed to indicate an adequate supply of gravel in an area where a remnant of an old gravel-capped plateau was known to exist and where gravel deposits seemed most likely to occur.

Digging test pits in sandy gravel formations is often hampered by serious caving within a few feet of the surface. Deep pits require relatively large openings and shoring of the walls for safety. Meeting these requirements is costly and time consuming and a more rapid and less expensive means of exploring for gravel is desired.

A number of routine field tests were made with the resistivity equipment in the area prospected by the shallow test pits to locate, if possible, the outlines of gravel deposits or pockets that might offer a saving in the construction work nearby. The results of these geophysical tests indicated that the earth resistivity method of test may be applied with considerable success in areas where a sandy gravel is underlain by a clayey sand, a sandy clay, or a clay stratum.

RESISTIVITY TESTS INDICATE LOCATION OF GRAVEL

The resistivity test, as the name implies, involves measuring the resistivity of the materials lying below the ground surface. The presence and approximate depth to any material having resistivity characteristics markedly different from that of the overlying or surrounding formations is revealed by variations in the resistivity values obtained. These are governed by the relative quantities of the materials involved.

The apparatus used by the Public Roads Administration is shown in figure 1.[2] The three units, the instrument containing the potential and current-measuring devices, the battery box and the reels of wire, can be readily carried over any ordinary terrain. The battery box, which contains sufficient batteries to supply approximately 200 volts to the current circuit, has compartments for carrying incidental equipment such as tools, tapes, electrodes, etc. The potentiometer, mounted on the left-hand portion of the instrument panel, is equipped with a sensitive galvanometer (0.125 microamperes per mm. on its own scale) and has a maximum range of 1.10 volts. The milliammeter, mounted on the right-hand portion of the panel, is a sensitive unit equipped with shunts to give full-scale readings of 0–15, 0–75, 0–150, and 0–300 milliamperes. The potential electrodes consist of porous porcelain pots, such as the one shown in figure 1, and they are filled with a saturated solution of copper sulphate in which is immersed a coil of heavy copper wire (No. 8 or larger). The current electrodes (not shown in figure 1) are heavy iron rods about 2 feet long and 1¼ inches in diameter.

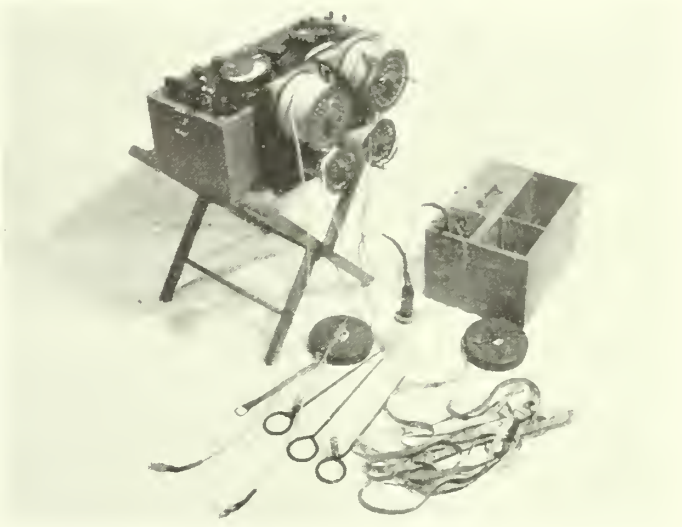


FIGURE 1.—RESISTIVITY APPARATUS USED BY THE PUBLIC ROADS ADMINISTRATION.

In making a resistivity test the apparatus is set up on an ordinary camp stool and the current electrodes are driven into the ground so spaced that one-third of the distance separating them constitutes the approximate desired depth of investigation. The two potential electrodes are placed at the third points between the current electrodes and all four electrodes are connected to the instrument by the wires provided for that purpose. A current of a few milliamperes strength is introduced into the ground, and a zero reading is obtained on the galvanometer of the potentiometer circuit by adjusting the potentiometer controls. The value of the potential drop set up in the potentiometer to balance that existing between the porous pots resting on the ground surface, together with the value of the current flowing through the ground as obtained from the milliammeter, are inserted in an appropriate formula and the apparent or average resistivity of the ground is obtained. The underlying theory and a diagram of the electrical connections are presented in the earlier report. The method has been referred to previously as the direct current method.

There are two tests commonly made with the resistivity apparatus, the step traverse and the depth test. In making a step traverse, resistivity data are obtained that relate to the formations in the subsurface within a fixed depth which is controlled by the spacing of the

[1] PROSPECTING FOR ROAD MATERIALS BY GEOPHYSICS. Stanley W. Wilcox. Engineering News Record, February 21, 1935, p. 271.

[2] *See Subsurface Exploration by Earth Resistivity and Seismic Methods*, by E. R. Shepard, published in the June 1935 issue of PUBLIC ROADS. Figure 4 of this earlier report shows the circuit used.

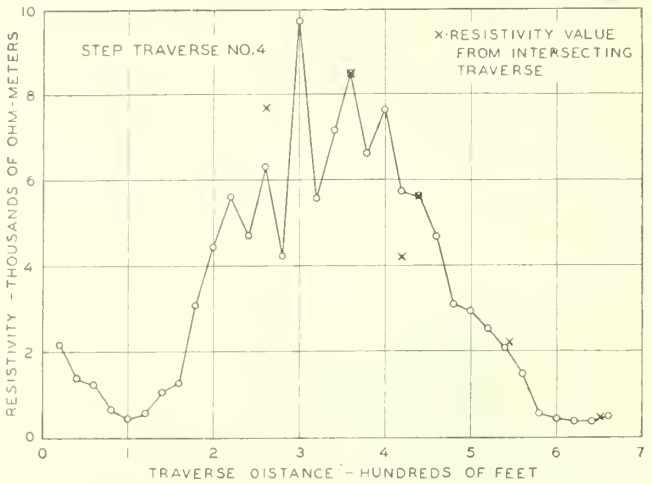

STEP TRAVERSE NO.4
×-RESISTIVITY VALUE FROM INTERSECTING TRAVERSE
RESISTIVITY - THOUSANDS OF OHM-METERS
TRAVERSE DISTANCE - HUNDREDS OF FEET

FIGURE 2.—STEP TRAVERSE OVER DEPOSIT OF SANDY GRAVEL WITH ELECTRODE SPACING OF 20 FEET.

electrodes on the ground surface. Experience has developed the empirical relation that the depth of the layer explored is approximately equal to the electrode spacing used. Thus, a 20-foot electrode spacing may be expected to yield information concerning a layer of material approximately 20 feet in depth. By maintaining this 20-foot spacing, and making tests at each 20-foot station or interval along a selected line of exploration, variations in resistivity are obtained which indicate increasing or decreasing amounts of material of high resistance, such as a sandy gravel, within the depth involved. If the resistivity values are plotted as ordinates against traverse distances as abscissas, the variations in the subsurface formations within the 20-foot layer are reflected with fair accuracy by variations in the resistivity graph. A step traverse over a deposit of sandy gravel underlain by clay and clayey sand is shown in figure 2.

When using this method of test in prospecting for gravel, the normal procedure is to make a series of tests along a random traverse line passing over likely locations. The proper electrode spacing is governed by the depth to which the information is desired. If a high resistivity zone is found (fig. 2), other tests are made on lines parallel to the random line at 50- to 100-foot intervals. Additional tests are made along lines, similarly spaced, but differing in orientation by approximately 90 degrees. This results in data for points that approximate a grid and it is possible to plot a resistivity contour map as shown in figure 3.

When a pocket of gravel is indicated, further exploration is made by depth tests, and it is desirable to make a few direct tests with a 6- or 8-inch post-hole auger. Samples of the material excavated from the bore holes show the nature of the material and can be used for determining the physical properties of the underlying formations.

In the depth test, data are obtained indicating the resistivities of the successive strata as one probes deeper into the ground. In this test the resistivity of the sub-

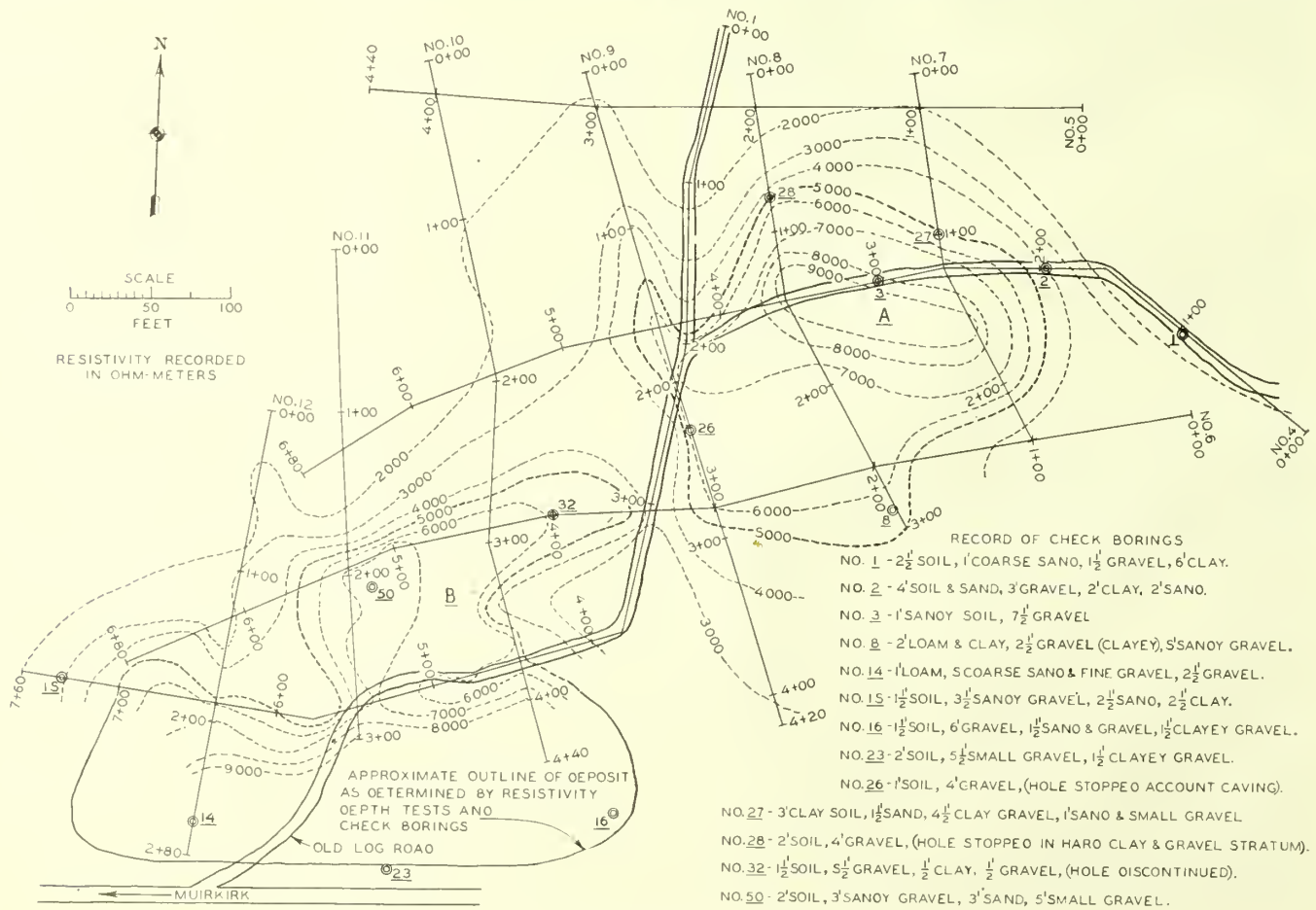

N
SCALE
0 50 100
FEET
RESISTIVITY RECORDED IN OHM-METERS
APPROXIMATE OUTLINE OF DEPOSIT AS DETERMINED BY RESISTIVITY DEPTH TESTS AND CHECK BORINGS
OLD LOG ROAD
MUIRKIRK
RECORD OF CHECK BORINGS
NO. 1 - 2½' SOIL, 1' COARSE SAND, 1½' GRAVEL, 6' CLAY.
NO. 2 - 4' SOIL & SAND, 3' GRAVEL, 2' CLAY, 2' SAND.
NO. 3 - 1' SANDY SOIL, 7½' GRAVEL
NO. 8 - 2' LOAM & CLAY, 2½' GRAVEL (CLAYEY), 5' SANDY GRAVEL.
NO. 14 - 1' LOAM, 5 COARSE SAND & FINE GRAVEL, 2½' GRAVEL.
NO. 15 - 1½' SOIL, 3½' SANDY GRAVEL, 2½' SAND, 2½' CLAY.
NO. 16 - 1½' SOIL, 6' GRAVEL, 1½' SAND & GRAVEL, 1½' CLAYEY GRAVEL.
NO. 23 - 2' SOIL, 5½' SMALL GRAVEL, 1½' CLAYEY GRAVEL.
NO. 26 - 1' SOIL, 4' GRAVEL, (HOLE STOPPED ACCOUNT CAVING).
NO. 27 - 3' CLAY SOIL, 1½' SAND, 4½' CLAY GRAVEL, 1' SAND & SMALL GRAVEL
NO. 28 - 2' SOIL, 4' GRAVEL, (HOLE STOPPED IN HARD CLAY & GRAVEL STRATUM).
NO. 32 - 1½' SOIL, 5½' GRAVEL, ½' CLAY, ½' GRAVEL, (HOLE DISCONTINUED).
NO. 50 - 2' SOIL, 3' SANDY GRAVEL, 3' SAND, 5' SMALL GRAVEL.

FIGURE 3.—RESISTIVITY CONTOUR MAP OF A DEPOSIT OF SANDY GRAVEL.

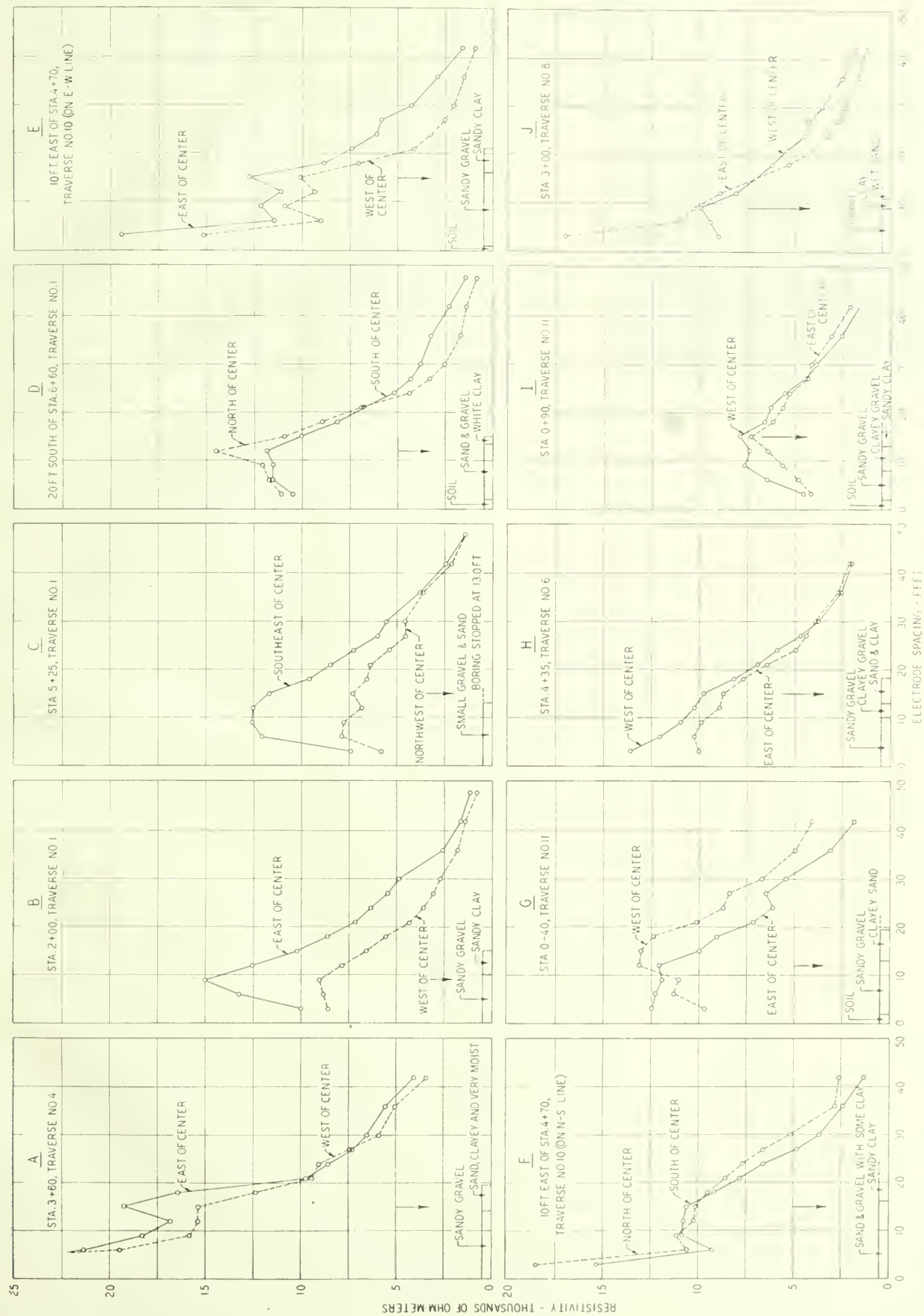
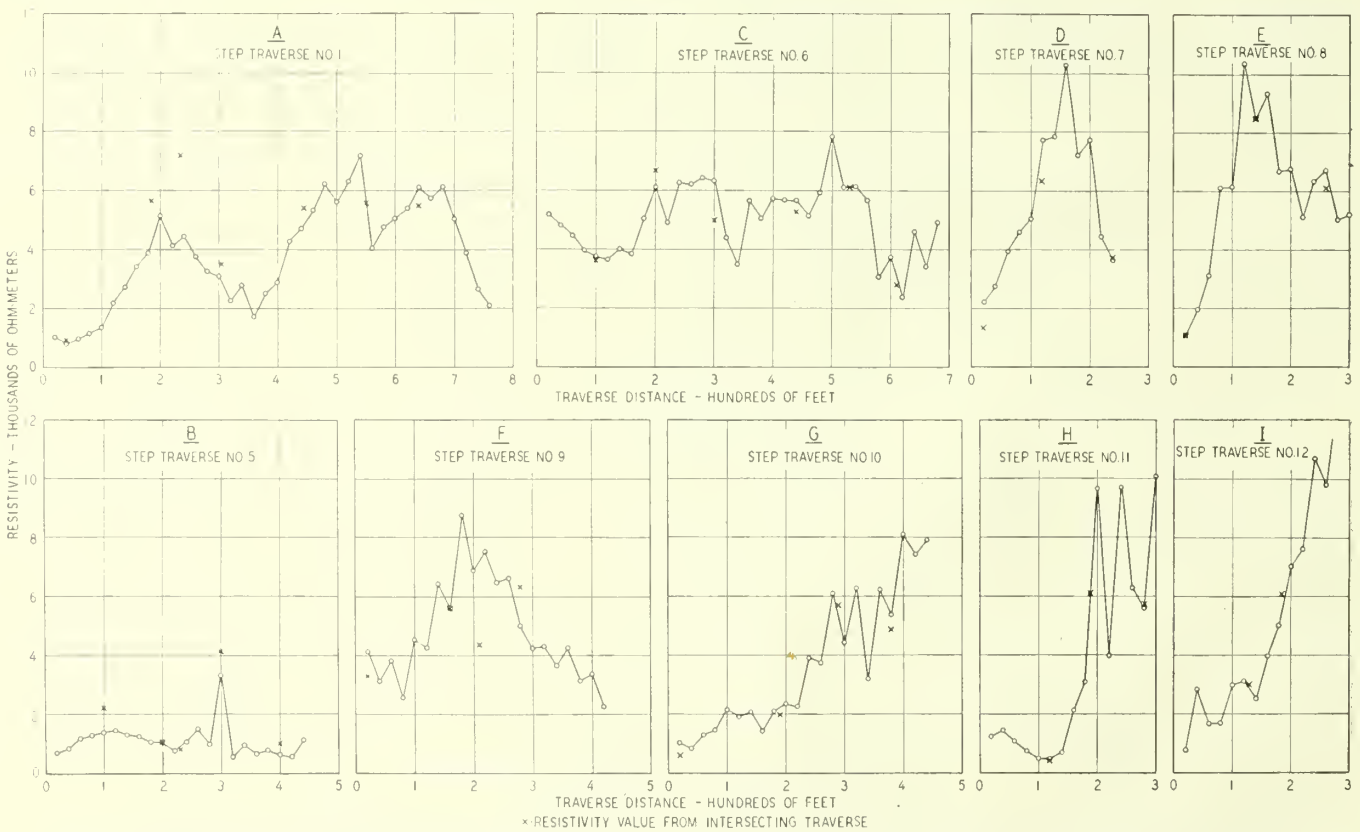


FIGURE 4.—RESULTS OF RESISTIVITY DEPTH TESTS IN A SANDY GRAVEL FORMATION UNDERLAIN BY CLAY OR CLAYEY SAND. TWO SETS OF READINGS WERE TAKEN AT EACH POINT, USING THE LEE PARTITIONING METHOD. RESULTS OF TEST BORINGS SHOWN IN LOWER LEFT CORNER OF EACH GRAPH.

FIGURE 5.—BORING WITH A POST-HOLE AUGER TO DETERMINE THE CHARACTER OF THE SUBSURFACE MATERIALS.

surface material is determined for a succession of gradually increasing electrode spacings. As the spacing is increased, the depth of material controlling the resistivity increases correspondingly. A common procedure in shallow exploration is to use an initial electrode spacing of 3 feet and subsequent spacings of 6 feet, 9 feet, 12 feet, etc. Each increase in the spacing of the electrodes probes approximately 3 feet deeper into the ground. The data are plotted with resistivity as ordinates and electrode spacing as abscissas and any inflection in the resistivity curve indicates the depth at which there is a change in the character of the formation. A number of such depth curves are shown in figure 4. In these tests there was a change from a gravel of high resistivity to clay or clayey sand with low resistivity. This change is indicated in the curves by the rather sharp decrease in resistivity at electrode spacings varying from 9 to 15 feet. In these tests the gravel lay within a few feet of the surface and the depth to the lower boundary of the gravel is indicated by the arrow appearing in each of the graphs. It is usually possible to pick the top horizon of the gravel also when the depth of the overburden is of sufficient importance to warrant it.

Borings made at the locations of several of the depth tests showed that this method of test and analysis indicated the lower horizon of the gravel rather accurately. Under favorable conditions holes 20 to 30 feet in depth may be bored with only moderate difficulty with an ordinary 6-inch post-hole auger (fig. 5). Two men may be expected to sink 4 or 5 holes to depths of 12 to



A - ALONG OLD LOG ROAD FROM A POINT ABOUT 400 FEET NORTH OF RIDGE TO A POINT ON SUMMIT AND TURNING APPROXIMATELY 100° WEST TO A POINT OVER THE RIDGE AND SOME 360 FEET DISTANT.
B - ABOUT 100 FEET NORTH OF TRAVERSE NO.4 - CROSSING NOS. 7, 8, 9, AND 10 AT STA. 0+20 AND TRAVERSE NO.1 AT 0+60 (0+60, NO.1 = 2+30, NO.5).
C - 100 FEET SOUTH OF TRAVERSE NO.4 - CROSSING NO.7 AT 2+40, NO.8 AT 2+60, NO.9 AT 2+80, NO.10 AT 2+90, NO.11 AT 1+90 AND NO.12 AT 1+30.
D - FROM A POINT 20 FEET NORTH OF STA. 1+00, NO.5 TO STA. 1+00 NO.6 STA. 1+20 NO.7 = 2+60, NO.4.
E - FROM A POINT 20 FEET NORTH OF STA. 2+00, NO.5, TO 40 FEET SOUTH OF STA. 2+00, NO.6 STA. 1+40, NO.8 = 3+60, NO.4
F - FROM A POINT 20 FEET NORTH OF STA. 3+00, NO.5, TO 140 FEET SOUTH OF 3+00, NO.6, - STA. 1+60, NO.9 = 4+40 NO.4, STA. 2+10 = 2+35 NO.1, STA. 2+80 = 3+00 NO.6.
G - FROM A POINT 20 FEET NORTH OF STA. 4+00 NO.5 TO A POINT 150 FEET SOUTH OF STA. 4+40, NO.6 STA. 1+90, NO.10 = 5+45 NO.4; STA. 2+90 NO.10 = 4+40, NO.6 STA. 3+80 NO.10 = 4+45 NO.1.
H - FROM A POINT 120 FEET NORTH OF NO.4 TO A POINT 20 FEET SOUTH OF NO.1 - STA. 1+20 = 6+50 NO.4 - STA. 1+90 = 5+30 NO.6, STA. 2+80 = 5+50 NO.1.
I - FROM A POINT 130 FEET NORTH OF NO.6 TO A POINT 75 FEET SOUTH OF NO.1 - STA. 1+30 = 6+10 NO.6 - STA. 1+85 = 6+40 NO.1.

FIGURE 6.—EFFECT OF GRAVEL DEPOSIT UPON THE RESISTIVITY TRAVERSE (OR CONSTANT DEPTH TEST) DATA WITH ELECTRODE SPACING OF 20 FEET.

15 feet in an 8-hour day provided excessively hard cemented strata are not encountered.

GRAVEL DEPOSITS SUCCESSFULLY LOCATED AT BELTSVILLE, MARYLAND

The tests were made at a point about 1.6 miles east of Muirkirk, Md., in a wooded area on the north side of the road which links Muirkirk with the Bowie road. A trial traverse was laid out along an old logging road passing through the area investigated. Resistivity tests were made at each 20-foot station along this line (traverse No. 1, fig. 3 and graph A, fig. 6). Two areas of high resistivity were located by this traverse. Another line of tests was run along another old road, crossing the first traverse near station 2+00 where the peak in the plotted graph occurs (traverse No. 4, fig. 3). A more positive indication of gravel was obtained in this second line of tests the results of which are plotted in figure 2. Referring to this figure, it will be seen that the resistivities encountered, involving as they do a zone extending approximately 20 feet to the right and to the left of the line of tests and some 20 feet in depth, were comparatively low for the first eight test points. A test boring at station 1+00 showed successively 2½ feet of sandy soil, 1 foot of coarse sand, 1½ feet of wet gravel, 5 feet of blue clay, and 1 foot of a heavy white clay. Other borings and resistivity depth tests made along this traverse showed a gradually increasing amount of gravel to be present as the peak of resistivity shown between stations 1+60 and 5+00 was approached. Beyond station 5+80 the tests indicate that the material within 20 feet of the surface is largely clay. Graphs A and B in figure 4 show comparisons of the results of borings with those of depth tests made at station 3+60 and at a point 10 feet south of station 4+20 (shown in graph B as station 2+00, traverse No. 1). The log of the boring is given in the lower left-hand corner of the graph. The close correlation between the resistivity depth test data and the data from borings is characteristic of all the tests made in the immediate vicinity. Graphs C to J show results of depth tests at other locations where check borings were made.

Only a few borings are normally required for use in interpreting the results of resistivity tests in a given area. A relatively large number were made in this study since there was no likelihood of being able to make a visual inspection of actual pit-face conditions at an early date and a direct and detailed check on the resistivity data was desired.

In a further expansion of the information concerning the gravel deposit other traverse lines were run parallel to the one shown in figure 2 (graphs B and C, fig. 6), and also a number of lines were run at approximately right angles to these lines and parallel to the one shown in graph A (see graphs D to I, fig. 6). Referring to figures 2 and 6, it is apparent that where one traverse crosses another there is usually similarity of the resistivity values obtained in the two separate test lines at their point of intersection. The plotted crosses indicate the values of resistivity obtained on the lines that crossed the traverse shown in the graph. In only one case, that where traverse No. 9 crosses No. 1, is there any considerable difference between the resistivity values obtained. It is believed that the information obtained with the resistivity apparatus is dependable when properly interpreted.

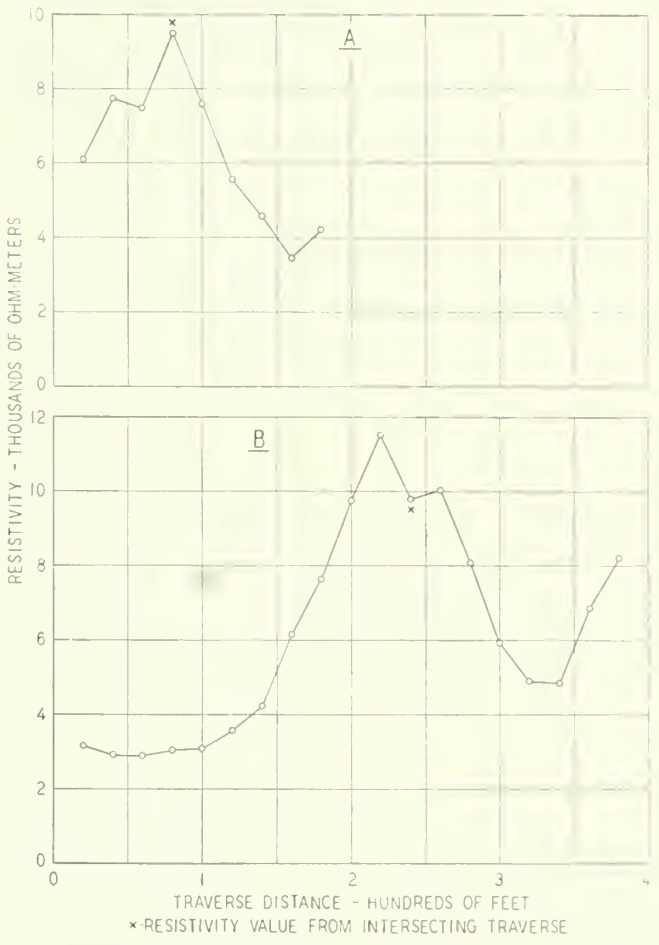


FIGURE 7.—DATA FROM TWO RESISTIVITY TRAVERSE LINES REQUIRED TO LOCATE AND OUTLINE A SMALL POCKET OF SANDY GRAVEL WITH ELECTRODE SPACING OF 20 FEET.

From the data obtained on the several traverse lines a resistivity contour map was made covering the entire area under investigation. This map is shown in figure 3 and it is used in somewhat the same manner as an ordinary surface contour map. The various contour lines represent resistivity and the area of highest resistivity is found to coincide with the thickest part of the gravel deposit.

Resistivity depth tests and check borings both indicated that an area roughly bounded by the 5,000-ohm-meter contour contained a gravel stratum averaging 8 to 10 feet in thickness. The data indicated also that the gravel would be found within the first 12 feet of depth over much of the area and a working face of 15 to 20 feet giving a fair mixture of sand, gravel, and clay for use in low cost road construction would be possible. The clay, for the most part, would be found in the lower 5 or 6 feet and in the form of clayey sand, sandy clay, or a comparatively pure clay.

An examination of the depth curves and data from test borings in figure 4 shows that the resistivity test data indicates the lower horizon of the gravel with a rather high degree of accuracy. The underlying material may vary from a sand with some clay particles to a heavy clay stratum and because of its variable nature it is not so readily defined. While a definite knowledge of the character of the underlying material may be of

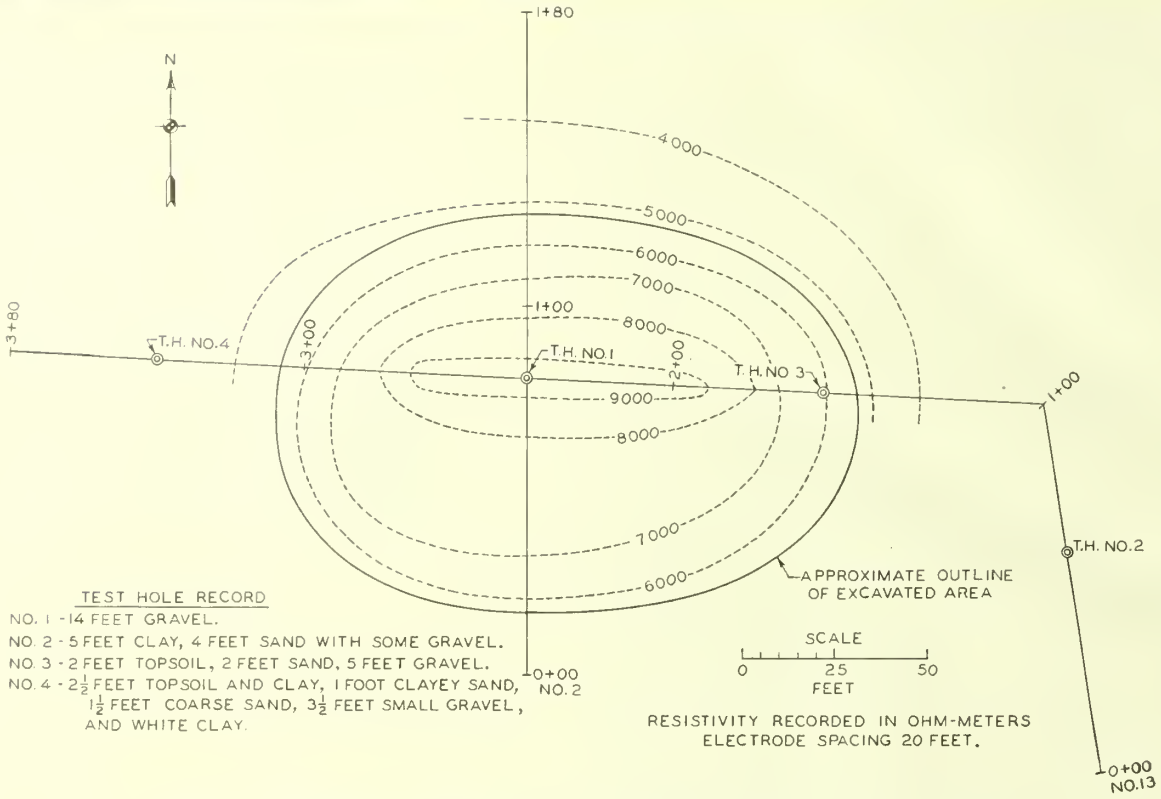


FIGURE 8.—RESISTIVITY CONTOUR MAP OF A SMALL POCKET OF SANDY GRAVEL.

value, in some instances, a rough estimate of the quantities of gravel available is usually of greater importance. It appears that estimates may be made with considerable confidence when based upon resistivity data augmented by the results of a few test borings.

From the data obtained from the tests that have been described, it is estimated that the area marked "A" in figure 3, should contain approximately 10,000 cubic yards of a sandy gravel with some clay and sand spots, all within about 15 feet of the surface. Area "B" is somewhat less uniform and should contain about 15,000 cubic yards of gravel.

Other tests made to the south of the area described in the preceding paragraphs, in an open field where some scattered excavations had already been made, indicated the presence of a pocket of gravel of considerable extent. A random traverse line was run (graph A, fig. 7) and a test boring made at the peak of the resistivity curve at station 0+80. Another traverse line was laid out approximately at right angles to the first and crossing it at the location of the test boring. Graphs A and B in figure 7 show the data from parts of the two traverse lines and figure 8 shows the resistivity contour map obtained from them.

Considering the data shown in figure 8, an area bounded by a contour between the 5,000- and 6,000-ohm-meter contours was selected as the approximate boundary of the most valuable portion of the pocket of gravel.

Subsequent to and because of the indications of the resistivity survey, the gravel pocket was excavated and about 6,000 cubic yards of material were removed. The solid line on the contour map of figure 8 gives approximately the outline of the area excavated.

CONCLUSIONS

The resistivity method of exploration as used in this study offers a rapid and inexpensive means of locating and delineating gravel deposits that are underlain by clay. The most useful field for this method of test is in rapid reconnaissance surveys over relatively large areas where surface indications are few or unreliable. Routine subsurface tests along a projected highway location may be used to locate small local deposits of road-building material satisfactory for construction purposes, thus eliminating the expense of transporting materials from more distant sources. In the work described above where 6,000 cubic yards of material was obtained, the necessary field work was completed by one engineer and two inexperienced helpers in about 8 hours. With the usual field party of two engineers and two experienced helpers a faster rate is possible.

While the resistivity method has proved to be entirely satisfactory in many instances in the past, it should be emphasized that this method of test is not always practical and that there are some field conditions under which resistivity tests will be of little value. A dry, loose overburden may possess such high resistivity as to completely mask the effect of sand, gravel, or rock in the deeper strata. Clay soils overlying or underlying clay-filled gravels may have resistivities so nearly the same as that of the gravel that it is difficult to interpret the resulting resistivity curves accurately. Formations that are inherently heterogeneous yield data that, for obvious reasons, are of little value.

In spite of these limitations, it is believed that the resistivity method of subsurface exploration has a definite field of usefulness in the solution of certain highway problems.

U. S. GOVERNMENT PRINTING OFFICE: 1944

STABILIZED BASE EXPERIMENT IN MINNESOTA

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by Henry Aaron, Highway Engineer[1]

CONSTRUCTION of satisfactory low-cost, all-weather roads in the Red River Valley of Minnesota has always presented a difficult problem. Loose sand and gravel placed on the heavy clay soils typical of the area had to be continually replaced as the material gradually penetrated into the subgrade to great depths and produced no apparent increase in stability.

The blotter type of bituminous surface was a notable improvement over loose gravel. It consisted of an application of a bituminous prime or penetration coat to the prepared subgrade, followed by a second application as soon as the first had dried. This was immediately covered with gravel in the amount of about 350 cubic yards per mile. Although this road surface resulted in the elimination of dust and the conservation of gravel, it did not prove entirely satisfactory under an increasing traffic.

The excellent performance of stabilized bases with thin bituminous wearing courses in other localities suggested a better method of overcoming difficulties. This led to the problem of required thickness of base. The Minnesota Department of Highways and the Public Roads Administration in 1936 initiated an experiment to determine by means of a service test the thickness of stabilized base required on the heavy clay subgrades of the Red River Valley.

Federal-Aid Project 349–A was selected for the experiment. The project, located on U. S. Highway 75, begins at the crossing of the tracks of the Great Northern Railroad about 2 miles north of Hendrum and extends east approximately 13.4 miles to the west city limits of Ada. The lay-out of the project is shown on figure 1.

Construction of the original base course was started in July and completed in September 1936. Construction of the bituminous wearing course was delayed until 1938, when it was necessary to recondition the base owing to the reduction in thickness and crown resulting from the action of traffic. The base course was reconstructed during August 1938 and the bituminous wearing surface consisting of a bituminous prime coat and a double blotter type sealing coat were placed in September 1938.

A traffic count made in 1941 showed that the road carried 394 vehicles per day (the 24-hour average) of which 14 percent were commercial vehicles such as light, medium, and heavy trucks and busses.

CONDITIONS ON PROJECT DESCRIBED

Conditions on this highway with respect to soils, drainage, surfacing materials, and climate are typical of the entire region of level plains adjacent to the Red River which forms the boundary between Minnesota and North Dakota. The ground surface slopes toward the Red River at the rate of only a few inches per mile.

Figure 1.—Location of Experimental Road and Sources of Materials.

The soils are heavy plastic clays or silty clays, commonly known as gumbo, and gravel for surfacing must be hauled for long distances.

Exceedingly low temperatures are recorded during the winter months and a heavy snowfall occurs. Owing to the flatness of the terrain, the road ditches become filled with water from melting snow and rain and remain that way for long periods in the spring.

In order to provide adequate surface drainage and also to facilitate snow removal, road grades are generally established a foot or more above the ground surface. Material for the fill is borrowed from alongside the road, resulting in wide bottom ditches 3 to 4 feet below the crown of the road.

A brief history of this section of road prior to the construction of the stabilized base will illustrate the performance of bituminous surfaces when placed directly on the clay subgrade.

In 1925 the roadway was graded to a width of 30 feet. In 1926 a blotter type bituminous surface was constructed, followed by re-treatments in 1927, 1928, 1929, 1931, and 1934. In 1936 only about 13 percent of the total length of the road was in good condition, while the remainder was rough as a result of cracking, breaking, pitting, scaling, and raveling. Typical failures are illustrated in figure 2.

DESCRIPTION OF EXPERIMENT

The experiment consisted of 16 test sections of stabilized base having thicknesses of 1, 2, 3, 4, 6, and 8 inches constructed on a roadbed which, as already described, was graded in 1925 and surfaced in 1926 with a bituminous-treated gravel. On four sections the base was placed directly on the existing bituminous surface. The bituminous mat was removed from the other sections before placing the base. The base on fourteen sections was a mixture of sand and gravel with binder soil and on two sections it consisted of sand and binder soil. Sand and gravel used in the mixtures were obtained from pits located in a ridge about 10 miles east of Ada. The gradations of the stabilized gravel mixtures were quite uniform as were those of the two stabilized sand mixtures. Variations in plasticity index were introduced by the use of binder soil obtained from

[1] Lieutenant, U. S. Navy, and former highway engineer, Public Roads Administration.

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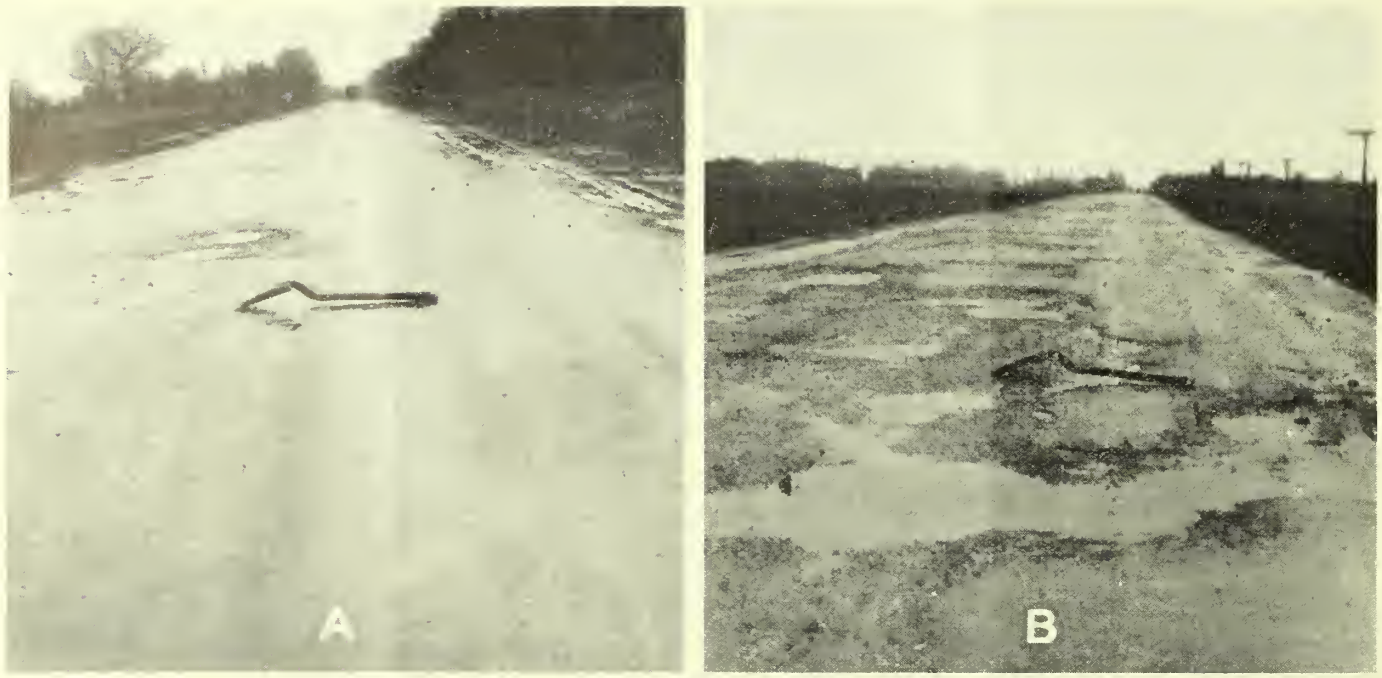
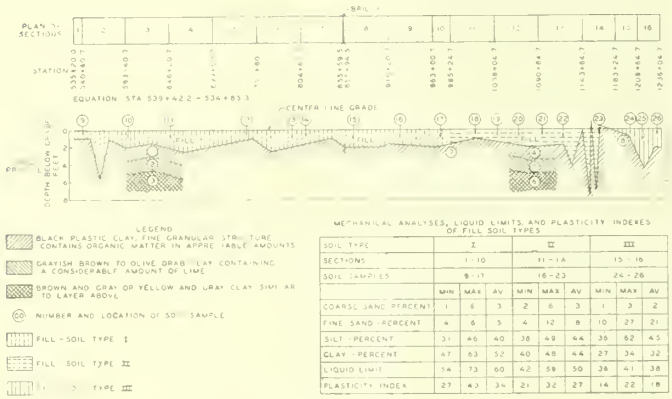


Figure 2.—Condition of Old Bituminous Mat, A, Typical of Better Portions of the Project and B, Typical of Sections Where Surfacing Was Completely Destroyed.


PLAN & SECTIONS
STATION
EQUATION STA 539+42.2 = 534+83.3
CENTER LINE GRADE
DEPTH BELOW GRADE
LEGEND
BLACK PLASTIC CLAY, FINE GRANULAR STRUCTURE CONTAINS ORGANIC MATTER IN APPRECIABLE AMOUNTS
GRAYISH BROWN TO OLIVE GRAY CLAY CONTAINING A CONSIDERABLE AMOUNT OF LIME
BROWN AND GRAY OR YELLOW AND GRAY CLAY SIMILAR TO LAYER ABOVE
NUMBER AND LOCATION OF SOIL SAMPLE
FILL - SOIL TYPE I
FILL - SOIL TYPE II
FILL - SOIL TYPE III

MECHANICAL ANALYSES, LIQUID LIMITS AND PLASTICITY INDEXES OF FILL SOIL TYPES

SOIL TYPE	I			II			III		
SECTIONS	1 - 10			11 - 14			15 - 16		
SOIL SAMPLES	9 - 17			18 - 23			24 - 26		
	MIN	MAX	AV	MIN	MAX	AV	MIN	MAX	AV
COARSE SAND - PERCENT	1	6	3	2	6	3	1	3	2
FINE SAND - PERCENT	4	6	5	4	12	8	10	27	21
SILT - PERCENT	31	46	40	38	49	44	35	62	45
CLAY - PERCENT	47	63	52	40	48	44	27	34	32
LIQUID LIMIT	56	73	60	42	59	50	38	41	38
PLASTICITY INDEX	27	43	34	21	32	27	14	22	18

Figure 3.—The Soil Profile.

various depths in three different pits. The distinguishing details of the various sections are given in table 1.

Table 1.—*Details of experimental sections*

Section number	Length	Thickness	Binder soil: Pit number	Binder soil: Limits from ground surface	Subgrade
	Feet	*Inches*		*Inches*	
1	998	1	1196	6–36	Soil.
2	5,280	4	1196	6–18	Do.
3	5,280	4	1196	18–36	Do.
4	5,280	4	1196	36–48	Do.
5	5,280	4	975	24–60	Do.
6	5,280	8	1196	18–48	Do.
7	5,099	6	1196	18–48	Do.
8	5,426	3	1196	18–48	Do.
9	5,280	2	1196	18–48	Do.
10	2,224	2	1195	18–54	Bit. mat.
11	5,280	3	1195	18–54	Do.
12	5,280	4	1195	18–54	Do.
13	5,280	6	1195	18–54	Do.
14	3,960	4	1195	18–36	Soil.
15[1]	2,640	6	1195	18–54	Do.
16[1]	2,640	3	1195	18–54	Do.

[1] Sections 15 and 16 are sand-clay while sections 1 to 14, inclusive, are sand-clay-gravel.

DESCRIPTION OF SUBGRADE SOILS

Subgrade soils on practically the full length of the project consist of fill material excavated from alongside the road. The height of fill and the soil profile are shown in figure 3. In general, the fill varies from 1 to 2.5 feet in height but reaches 5 to 8 feet at a few places where the road crosses drainage ditches and small streams.

The soil profile, typical of the area, consists of a layer of black plastic clay containing organic matter in

Table 2.—*Mechanical analyses and physical properties of subgrade soils*

Sample number	Mechanical analysis: Coarse sand (2 to 0.25 mm.)	Mechanical analysis: Fine sand (0.25 to .05 mm.)	Mechanical analysis: Silt (0.05 to .005 mm.)	Mechanical analysis: Clay (smaller than 0.005 mm.)	Physical constants: Liquid limit	Plasticity index	Shrinkage limit	Shrinkage ratio	Centrifuge moisture equivalent	Field moisture equivalent
	Percent	*Percent*	*Percent*	*Percent*						
1	2	6	48	44	65	36	12	1.9	45	32
2	1	3	43	53	62	36	17	1.8	37	29
3	1	5	44	50	54	34	19	1.7	32	35
4	7	24	(1)	(1)	66	29	18	1.7	39	44
5	3	13	(1)	(1)	43	19	20	1.8	25	26
6	2	7	(1)	(1)	49	25	20	1.7	28	26
7	3	11	48	38	61	32	17	1.7	37	39
8	1	29	53	17	43	12	18	1.7	26	32
9	2	4	31	63	73	43	23	1.6	42	29
10	2	5	40	53	61	36	13	1.9	37	32
11	1	4	42	53	63	38	17	1.8	28	26
12	3	6	44	47	56	34	15	1.9	33	30
13	6	6	34	54	58	30	15	1.9	41	32
14	5	6	39	50	54	30	13	1.9	35	28
15	4	4	43	49	62	36	12	1.9	35	29
16	3	4	46	47	54	27	13	1.9	32	27
17	5	5	38	52	57	30	12	2.0	34	29
18	2	7	48	43	42	21	14	1.9	27	24
19	2	7	49	42	47	25	17	1.9	30	27
20	4	4	44	48	52	30	14	1.9	32	26
21	6	8	38	48	44	23	14	1.8	29	24
22	4	11	45	40	59	32	16	1.8	38	35
23	2	12	41	45	58	30	13	1.9	34	35
24	2	26	38	34	41	22	14	1.9	27	23
25	1	10	62	27	36	14	20	1.8	26	24
26	3	27	36	34	37	19	13	1.9	25	20

[1] Flocculated.

TABLE 3.—*Maximum dry densities and optimum moisture contents of samples of soil of each type*

Sample number	Type of soil	Dry density	Optimum moisture
		Lb. per cu. ft.	*Percent*
10	I	99	22
20	II	105	20
26	III	110	17

TABLE 4.—*Average densities and moisture contents of samples of each type of soil*

Sections	Type of soil	Dry density	Average moisture content
		Lb. per cu. ft.	*Percent*
1–10	I	92	27
11–14	II	105	22
15–16	III	102	22

appreciable amounts. This layer extends to a depth of 18 inches and is underlaid by a grayish-drab colored clay rich in lime. At a depth of about 36 inches, the soil changes to a mottled brown and gray or yellow and gray clay.

Material from the two upper layers of the soil profile was used in constructing the fill. As a result, the subgrade soil at any particular location consists of either the top layer, the second layer, or a mixture of the two layers. According to the results of tests, table 2, the soils in the fill as well as those representing the different layers of the soil profile have physical properties of the A–6 and A–7 groups with the lighter textured, less plastic varieties near the eastern end of the project in sections 15 and 16.

The subgrade soils in sections 1 to 10 were considerably more plastic and heavier textured than those in sections 15 and 16 while the fills in sections 11 to 14 were composed of materials intermediate between these two types. For this reason, the soils in the fills were divided into three types as shown in figure 3.

Samples 10, 20, and 26 (table 2) are typical of soil types I, II, and III, respectively. The results of standard compaction tests performed on these three samples are shown graphically in figure 4. Maximum dry densities and optimum moisture contents are shown in table 3.

The actual densities of the upper 12 inches of the subgrade at the locations of samples 10, 20, and 26 were respectively, 95, 104, and 106 pounds per cubic foot with corresponding moisture contents of 27, 22, and 20 percent. These values agree quite closely with the average densities and moisture contents determined prior to the construction of the base course as shown in table 4.

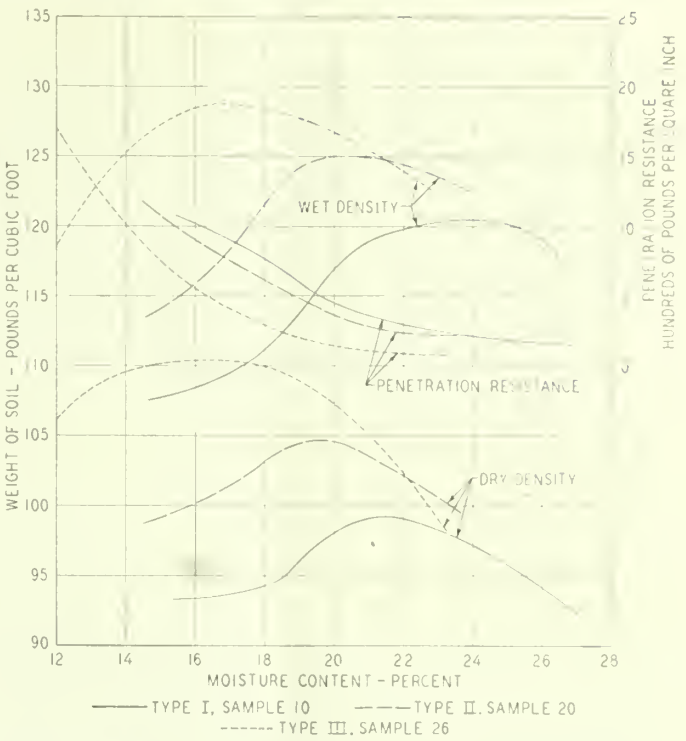

WEIGHT OF SOIL - POUNDS PER CUBIC FOOT
PENETRATION RESISTANCE HUNDREDS OF POUNDS PER SQUARE INCH
MOISTURE CONTENT - PERCENT
WET DENSITY
PENETRATION RESISTANCE
DRY DENSITY
TYPE I, SAMPLE 10
TYPE II, SAMPLE 20
TYPE III, SAMPLE 26

FIGURE 4.—TYPICAL DENSITY AND PENETRATION CURVES.

BASE-COURSE MATERIALS

The materials used in the stabilized mixtures were obtained from various pits located as shown on figure 1. Pits 975, 1195, and 1196 furnished the binder soils. The material from pit 975 was a clay till of glacial origin comparable to binder soils available in other sections of the State. The soils in pits 1195 and 1196 were lacustrine in origin and more or less typical of the soils found throughout the valley.

Results of mechanical analyses and physical and chemical tests performed on the different binder soils are given in table 5. Pit 1195 furnished a heavy clay with a considerable amount of both organic matter and lime. The material obtained from a depth of 6 to 18

TABLE 5.—*Results of tests on binder soils*

Pit number	Depth	Mechanical analysis				Physical characteristics of material						Chemical analysis					
		Coarse sand (2 to 0.25 mm.)	Fine sand (0.25 to 0.05 mm.)	Silt (0.05 to 0.005 mm.)	Clay (smaller than 0.005 mm.)	Liquid limit	Plasticity index	Shrinkage limit	Shrinkage ratio	Centrifuge moisture equivalent	Field moisture equivalent	Organic matter	CaO	MgO	CO_2	SiO_2	$Fe_2O_3+Al_2O_3$
	Inches	*Percent*	*Percent*	*Percent*	*Percent*							*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
975	24–60	3	12	35	50	42	25	14	1.9	26	23	1.23	12.3	5.2	12.8	19.2	13.9
1195	18–36	3	13	(1)	(1)	43	19	20	1.8	25	26	9.23	16.1	6.1	14.6	40.6	11.1
1195	18–54	2	7	(1)	(1)	49	25	20	1.7	27	26	1.05	11.3	6.6	13.9	16.6	12.4
1196	6–36	1	4	12	53	63	38	17	1.8	36	38	2.00	9.0	5.4	10.1	30.7	27.2
1196	6–18	2	6	18	44	65	36	12	1.9	45	32	1.59	1.1	3.4	.9	63.8	17.9
1196	18–36	1	3	13	53	62	36	17	1.8	37	29	1.15	9.8	6.2	11.6	48.7	16.3
1196	36–48	1	5	14	50	51	34	19	1.7	32	35	2.06	7.7	6.2	10.1	42.8	16.3
1196	18–48	1	3	32	64	60	33	17	1.8	37	35	0	9.7	6.3	11.4	49.3	16.7

[1] Flocculated.

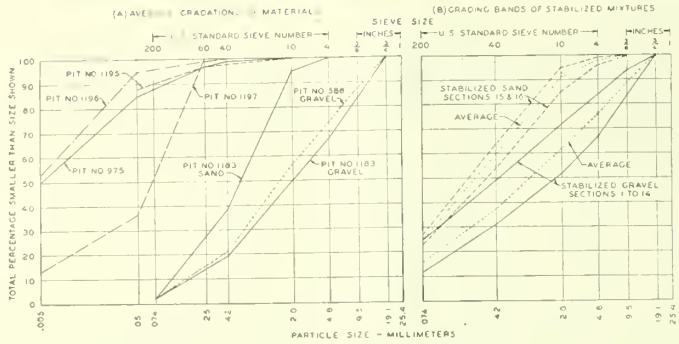


Figure 5.—Gradations of Materials and Mixtures Used in Original Construction.

inches in pit 1196 was high in organic matter and low in lime, while the other binder soils from this pit were heavy clays containing appreciable amounts of lime and relatively low percentages of organic matter.

Gravel for test sections 1 to 14 was produced from pits 1183 and 588, and the sand for sections 15 and 16 was screened from the gravel in pit 1183. Since the gravel from pits 1183 and 588 was deficient in fine sand and very fine sand, a fine sandy loam obtained from pit 1197 was added to the mixtures on all sections to improve the grading. This material also reduced the plasticity index of the highly plastic binder soils.

Average gradings of the sand, gravel, fine sandy loam, and binder soils are shown in table 6 and figure 5, *A*.

Table 6.—*Average gradations of materials used in stabilized mixtures*

Material	Pit number	Percentage smaller than—								
		¾-inch sieve	⅜-inch sieve	No. 4 sieve	No. 10 sieve	No. 40 sieve	No. 60 sieve	No. 200 sieve	0.05 mm.	0.005 mm.
Gravel	588	100	88	74	57	21	.....	2	.....	.....
Do.	1183	100	84	67	50	19	.....	2	.....	.....
Sand	1183	.....	.....	100	95	38	.....	2	.....	.....
Do.[1]	1197	.....	.....	.....	.....	100	99	.....	36	13
Binder	975	.....	.....	.....	100	99	97	.....	85	50
Do.	1195	.....	.....	.....	100	98	97	.....	88	.....
Do.	1196	.....	.....	.....	.....	100	99	.....	95	53

[1] Fine sandy loam.

DESIGN OF EXPERIMENTAL SECTIONS

Typical base-course cross sections are shown in figure 6. Sections 1, 2, and 3 inches thick were constructed in a single layer; sections 4 inches in thickness in two 2-inch layers; sections 6 inches thick in two 3-inch layers; and the 8-inch section was composed of two layers 3 inches thick and one layer having a thickness of 2 inches.

On all sections consisting of more than one layer, the stabilized mixtures for all but the top layer were proportioned in accordance with the percentages by weight shown in table 7.

Flake calcium chloride was added to all sections consisting of a single layer and to the surface layer of those sections having more than one layer. The proportions by weight are shown in table 8.

Sections 15 and 16 consisted of mixtures of sand combined with the various other materials while gravel was used in sections 1 to 14. Section 5 differs from the other gravel sections in that the binder soil used in the mixture was a glacial till from pit 975. Sections 1 to 4 and 6 to 14 contained binder soil high in organic matter and lime from pits 1195 and 1196.

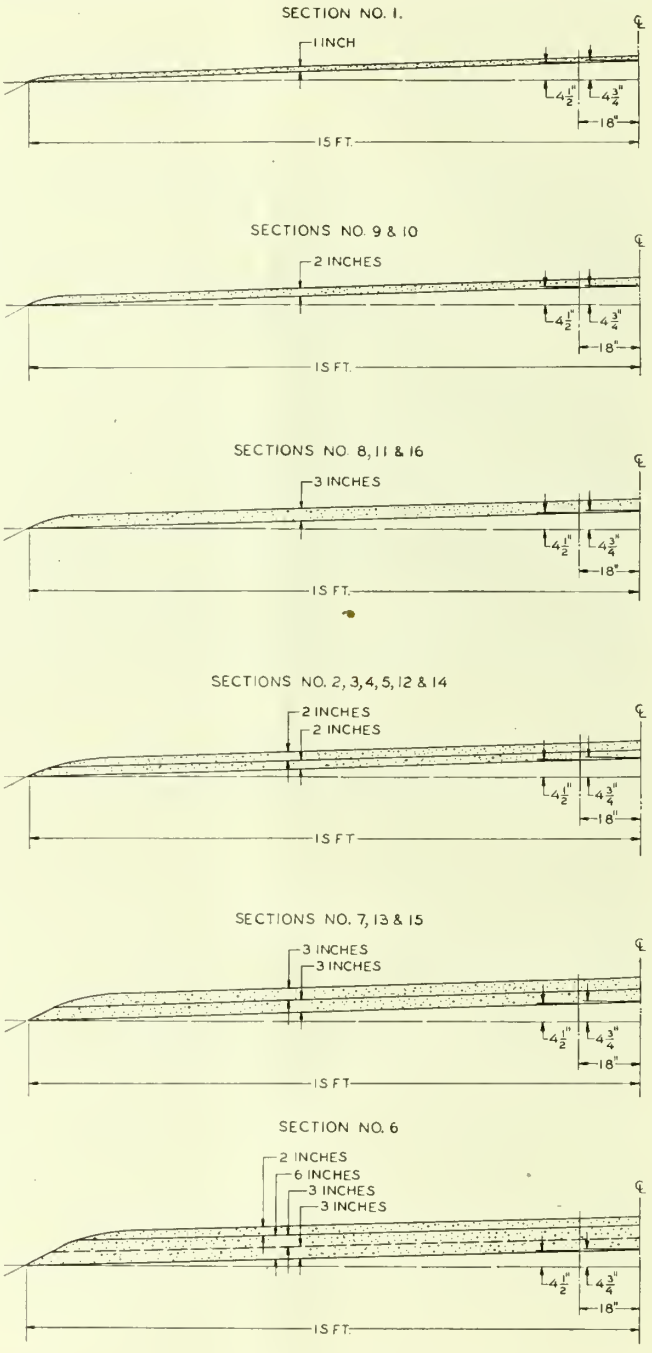


Figure 6.—Typical Cross Sections of the Stabilized Gravel Base. On All Sections the Center 3 Feet of the Roadway is to be Crowned One-Quarter of an Inch. The Slope From 18 Inches Right and Left of the Center Line is to be One-Third of an Inch per Foot.

Table 7.—*Percentages of materials by weight for all but the top layer on all multilayer sections*

Material	Sections 2, 3, 4, 6, 7, 12, 13, 14	Section 5	Section 15
	Percent	*Percent*	*Percent*
Gravel or sand	74.25	73.00	62.00
Binder soil	10.00	13.00	13.00
Fine sandy loam	7.25	5.50	16.50
Water	8.50	8.50	8.50

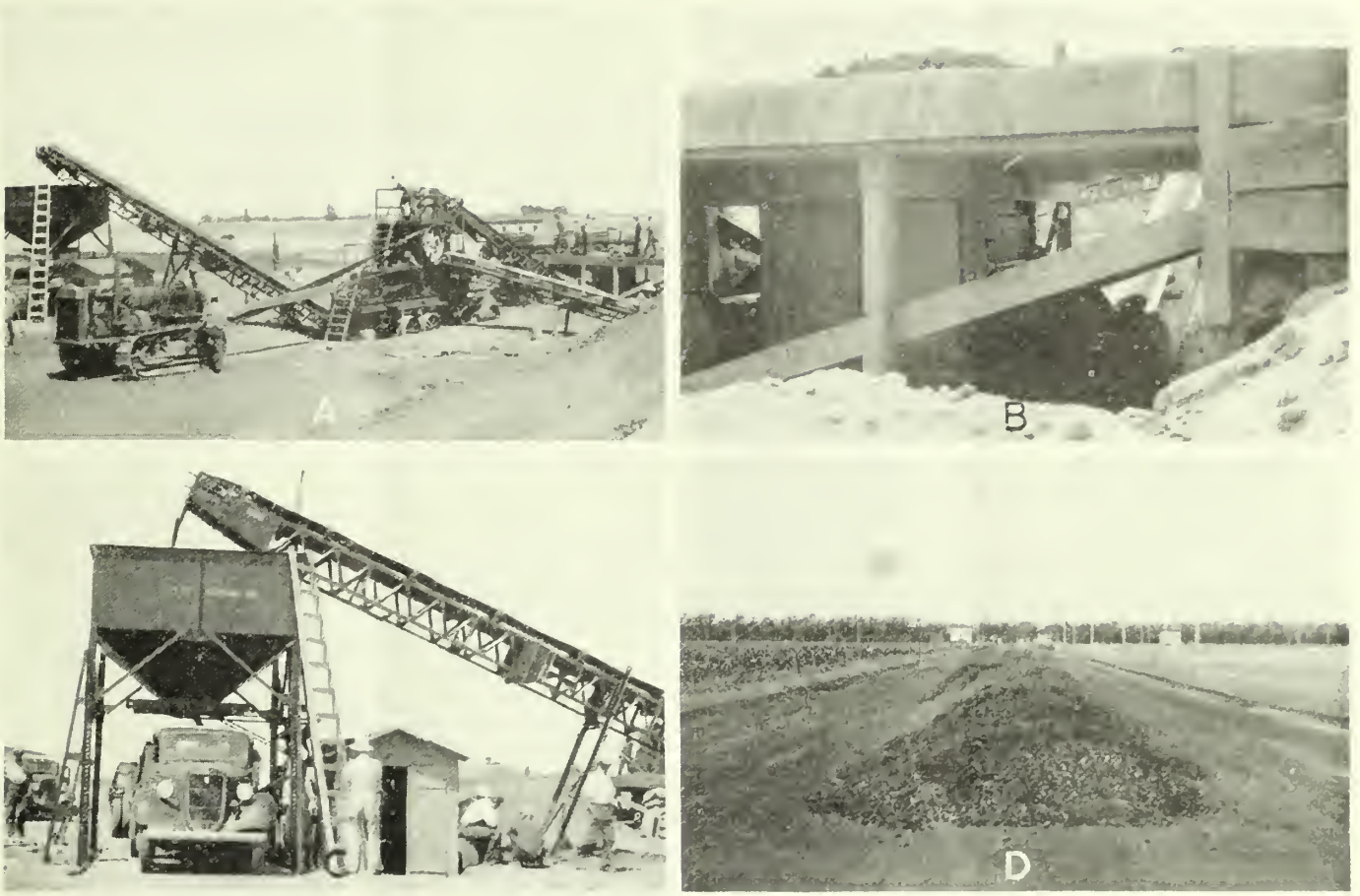


Figure 7.—Mixing Equipment: A, Central Mixing Plant, B, Loading Trap and Conveyor for Delivering Binder Soil to Shredder, C, Truck Resting on Platform Scale Being Loaded with Stabilized Mixture, and D, the Mixture Deposited in Windrow in Center of Road.

Table 8.—*Percentages of materials by weight in sections consisting of a single layer and in top layer of multilayer sections*

Material	Sections 1 to 4 and 6 to 14	Section 5	Sections 15 and 16
	Percent	*Percent*	*Percent*
Gravel or sand	74.00	72.75	61.75
Binder soil	9.90	12.90	12.90
Fine sandy loam	7.15	5.40	16.40
Water	8.50	8.50	8.50
Calcium chloride	.45	.45	.45

CONSTRUCTION METHODS

On all sections, exclusive of 10, 11, 12, and 13, the existing bituminous mat was scarified and removed from the road by blading it beyond the shoulders and was later used for dressing the shoulder slopes. The subgrade was then bladed to the typical cross section shown in figure 6 and rolled.

On sections 10, 11, 12, and 13, the old bituminous mat was left in place and the gravel placed directly on it. However, it was necessary to make many patches and blade the surface during hot weather in order to remove irregularities. Although the mat was in fairly good condition prior to placing the stabilized gravel, it was impossible to shape the surface to the desired section.

The different materials used in the stabilized mixtures were stock piled in a central mixing plant at Ada where they were combined (fig. 7, *A*). The plant consisted of a portable single pugmill continuous mixer equipped with a clay feeder; clay shredder; spray pipes for adding water; hoppers with adjustable gates for feeding the proper amounts of mineral salts, sand, and gravel; and belt conveyors for introducing the different materials into the mixer and discharging the mixture into a storage bin. Power was furnished by a 60-horsepower tractor. The output of the plant varied from 65 to 95 tons of mixed material per hour.

In the operation of the plant a controlled amount of clay was fed by screw conveyors to a belt conveyor which introduced the clay into the clay shredder at the top of the plant where it was pulverized before entering the pugmill. Figure 7, *B* shows the trap through which the binder soil was loaded into the clay feeder and the belt conveyor for delivering the soil to the shredder.

The same belt conveyor which elevated the binder soil was also used to deliver sacks of calcium chloride to the salt hopper at the top of the plant, from which it was fed at a controlled rate through an adjustable gate into the pugmill.

Simultaneously with the introduction of the binder soil, calcium chloride, and a regulated amount of water, the sand and gravel were introduced on another belt conveyor. The mixed material was discharged from the pugmill onto a third belt conveyor which carried the mixture to a storage bin located directly over a weighing platform.

Proportioning of the various materials was controlled by regulating the rate of feed to the mixer. First the speeds of the conveyor belts were determined while the plant was running at its normal operating speed. Then

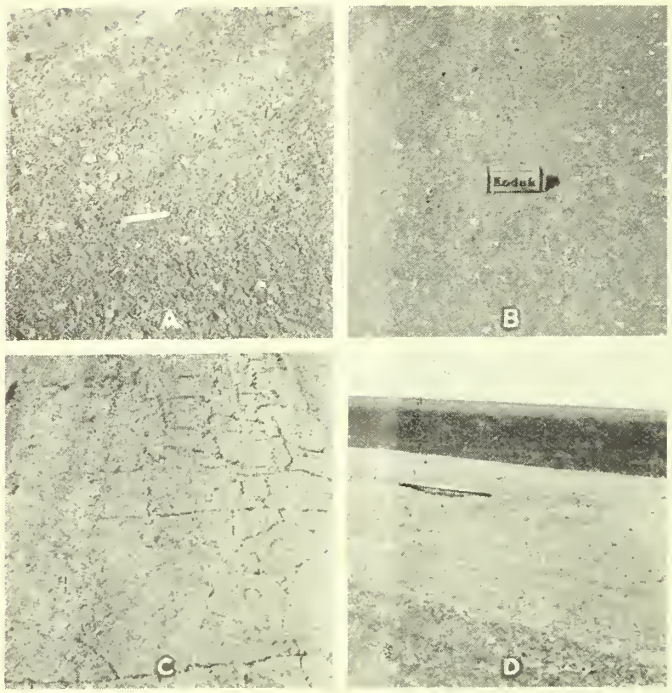


Figure 8.—Surface During Construction Showing: A, Appearance of Stabilized Mixture Prior to Rolling, B, Surface Texture After Compaction, C, Surfacing Displaced on Section 1, and D, Corrugations Produced by Traffic During Hot, Dry Weather, Characteristic of the Construction Period.

the plant was stopped and the weight of material on a 10-foot section of each belt was measured. From these two determinations the amount of each material delivered to the pugmill in a given time was calculated. Gate openings on the material hoppers were then adjusted until the desired proportions were obtained.

A final check on the proportioning was made by taking samples of the mixture discharged from the pugmill and determining their gradations, physical constants, and moisture contents.

The stabilized mixture was loaded from the storage bin into trucks resting on platform scales (fig. 7, *C*) and hauled out and deposited in the center of the road as shown in figure 7, *D*.

Distribution of the stabilized material on the road was controlled on the basis that the mixture would have a dry weight of 140 pounds per cubic foot after compaction. All trucks were loaded with 4.2 tons of the mixture and the loads were deposited a distance apart determined according to the compacted thickness of the layer to be constructed. The spacing of the loads of 4.2 tons varied from 22 feet for a 1-inch layer to 7.3 feet for a 3-inch layer.

The deposited material was spread and shaped to the desired cross section by a motor grader and compacted by a multiwheel, pneumatic tire roller (see cover illustration). The total weight of the roller and surcharged load was about 6 tons. Rolling was continued until a uniformly dense surface was obtained.

The appearance of the spread material before rolling is shown in figure 8, *A*. Figure 8, *B* shows the surface texture after final rolling.

Following the completion of sections 1 to 5, it was observed that a great deal of loose material developed under the action of traffic. It was therefore decided to apply calcium chloride to the surface in addition to that used in the mix. The application was made at the rate of 0.5 pound per square yard either after a rain or after sprinkling.

Beginning with section 6 and continuing to the end of the project, 0.25 pound per square yard was applied over the entire surface immediately after shaping and prior to the final rolling. An additional 0.25 pound per square yard was later spread over the central 24 feet following a rain or after sprinkling with 2,000 to 3,000 gallons of water per mile.

Section 1 was originally constructed with a thickness of 1 inch. Within 2 weeks after completion the surface became loose and raveled. Over large portions, the entire surface was worn away exposing the clay subgrade. This is illustrated in figure 8, *C*. Instead of attempting to maintain this section at its original thickness, it was reconstructed and the thickness increased to 4 inches. The new material was the same as that used in section 6. It was evident that a stabilized course 1 inch thick could not be satisfactorily maintained as a wearing surface when resting on the heavy clay subgrade typical of this project.

The construction period, July 11 to September 15, 1936, was unusually hot and dry. The surface dried out rapidly and became loose and corrugated (fig. 8, *D*). The first rains occurred on August 13 and 14 and soaked through the loosened material, softening the surface and resulting in a great deal of rutting as shown in figure 9, *A* and *B*. The rains proved beneficial, however, making it possible to remove all surface irregularities with motor graders. All the sections were maintained thereafter in good condition by light blading. Figure 9, *C* shows the general condition of the road at the time of completion of the project.

STABILIZED MIXTURES UNIFORMLY GRADED

Tests on samples taken from the different sections (table 9) show that the stabilized mixtures were well graded and that gradation of the mixtures was very uniform in all sections. Ranges in gradations are shown graphically in figure 5, *B*.

The uniformity of the mixtures indicates the efficiency of the methods of mixing and control adopted on this project. Study of the performance of the different sections is not complicated by lack of uniformity in the mix.

Table 9.—*Average gradations and physical properties of stabilized mixtures*

Section number	Gradation						Ratio B/A	Liquid limit	Plasticity index
	Percentage passing sieve indicated								
	¾-inch	⅜-inch	No. 4	No. 10	No. 40 (A)	No. 200 (B)			
1	100	84	71	57	37	15	0.41	23	7
2	100	89	76	64	39	17	.44	21	6
3	100	92	76	62	42	20	.48	23	8
4	100	91	80	64	35	16	.46	24	9
5	100	86	78	64	39	16	.41	18	4
6	100	91	81	68	41	18	.44	25	10
7	100	88	74	60	36	15	.42	22	8
8	100	89	72	60	39	18	.46	23	9
9	100	85	71	55	31	13	.42	23	8
10	100	89	77	64	39	18	.46	20	4
11	100	88	75	63	37	15	.41	18	4
12	100	86	74	58	32	13	.41	19	4
13	100	87	70	57	34	14	.41	18	4
14	100	91	76	62	36	14	.39	18	4
15	100	97	90	59	25	42	18	4	
16	100	99	95	62	27	.44	19	4	

Figure 9.—Conditions of Surface Showing: A, Typical Section Completed 3 Weeks or More Before the Heavy Rain, B, Completed 2 Days Before the Heavy Rain, C, General View at the Time of Completion of the Project, and D, Surface Raveling.

Table 10.—*Comparison between liquid limits and plasticity indexes of binder soils and of stabilized mixtures*

Pit number	Liquid limit range		Plasticity index range	
	Raw soil	Mixture	Raw soil	Mixture
975	42	18	25	4
1195	43–49	18–20	19–25	4
1196	54–65	21–25	33–38	6–10

PHYSICAL PROPERTIES OF MIXTURES INFLUENCED BY BINDER SOIL

Since gradations of material in the stabilized mixtures are uniform, the variation in physical properties may be correlated with the characteristics of the binder soil. Table 10 shows that variations in the liquid limit and plasticity index of the binder soils are definitely reflected in the characteristics of the mixtures. The data of table 10 show further, when considered with the data of table 5, that the liquid limit and plasticity index of the mixture are not influenced by the origin of the binder soil used nor by its chemical composition, except insofar as these may affect the plasticity of the binder soil.

MEASUREMENTS OF DENSITY INDICATE UNIFORM COMPACTION

As the construction of the various sections was completed, density measurements were made on the layers

Table 11.—*Results of density determinations*

Section number	Design thickness	Number of days[1] after construction	Average densities		Remarks
			Upper 2 to 3 inches	Bottom layer	
	Inches		*Lb. per cu. ft.*	*Lb. per cu. ft.*	
1	1	2	128	.....	Tests at 50 days made on reconstructed section 4 inches thick.
		50	140	.....	
2	4	1	.....	130	Test on entire thickness gave 137 pounds per cu. ft.
		30	134	.....	
		70	139	.....	
3	4	24	136	.....	
		64	140	.....	
4	4	22	134	.....	
		60	137	.....	
5	4	16	141	.....	
		80	139	.....	
6	8	15	135	130	
		50	138	131	
7	6	5	130	130	
		46	136	133	
8	3	5	136	.....	
		41	138	.....	
9	2	42	140	.....	
10	2	40	142	.....	
11	3	38	141	.....	
12	4	35	136	.....	
13	6	31	137	134	
		63	137	.....	
14	4	28	141	137	
15	6	26	130	132	Sand section
		60	134	134	
16	3	26	137	.....	Sand section
		60	137	.....	

[1] Approximate number of days from date of completion to date tests were made.

Figure 10.—Views of the Road Surface With: A, Entire Surface Displaced and Mat Exposed on Section 10, B, Scaling Condition, C, Shrinkage Cracks, and D, Pitting.

of the compacted stabilized mixtures. These measurements were later repeated to determine the effect of traffic on density. The average densities recorded on each section are given in table 11. The data for the gravel sections are summarized in table 12.

These results indicate uniform compaction throughout the length of the gravel portion of the project and a definite increase in compaction under the action of traffic during the first month following construction with practically no increase thereafter. The same is true for the stabilized sand mixtures. The sand sections, however, had lower average densities than the gravel sections.

Table 12.—*Range in average density of gravel sections after different periods under traffic*

Number of days under traffic	Upper 2 to 3 inches	Bottom layer
	Lb. per cu. ft.	*Lb. per cu. ft.*
5 days or less	128 to 136	130
15 to 31 days	134 to 141	130 to 137
35 to 80 days	136 to 142	131 to 137

BEHAVIOR OF STABILIZED GRAVEL UNDER TRAFFIC

The stabilized gravel was maintained under traffic for approximately 2 years prior to the construction of the bituminous surface. Condition surveys made during this period furnished information relative to the behavior of the various sections when used as road surfaces. The results of the condition surveys are summarized in table 13.

Formation of loose material on the surface as a result of the abrasive action of traffic was in evidence on all sections. Other surface defects formed such as corrugations, raveling, pitting, scaling, potholes, shrinkage cracks, and wearing out of the entire thickness of the stabilized surface. Figure 8, *D* shows examples of corrugations.

Raveling denotes a progressive breaking up of the surface resulting from the displacement of loosened material. It differs from the ordinary development of float on the surface in that loosening and displacement of measurable thicknesses occur in localized areas (fig. 9, *D*). In some places, raveling progressed to the extent that the entire surface was worn off (fig. 10, *A*). This has been recorded in table 13 as exposed subgrade, or exposed bituminous mat on the sections where the old bituminous mat was not removed prior to stabilization.

Scaling (fig. 10, *B*) refers to the condition where the surface peels off in layers.

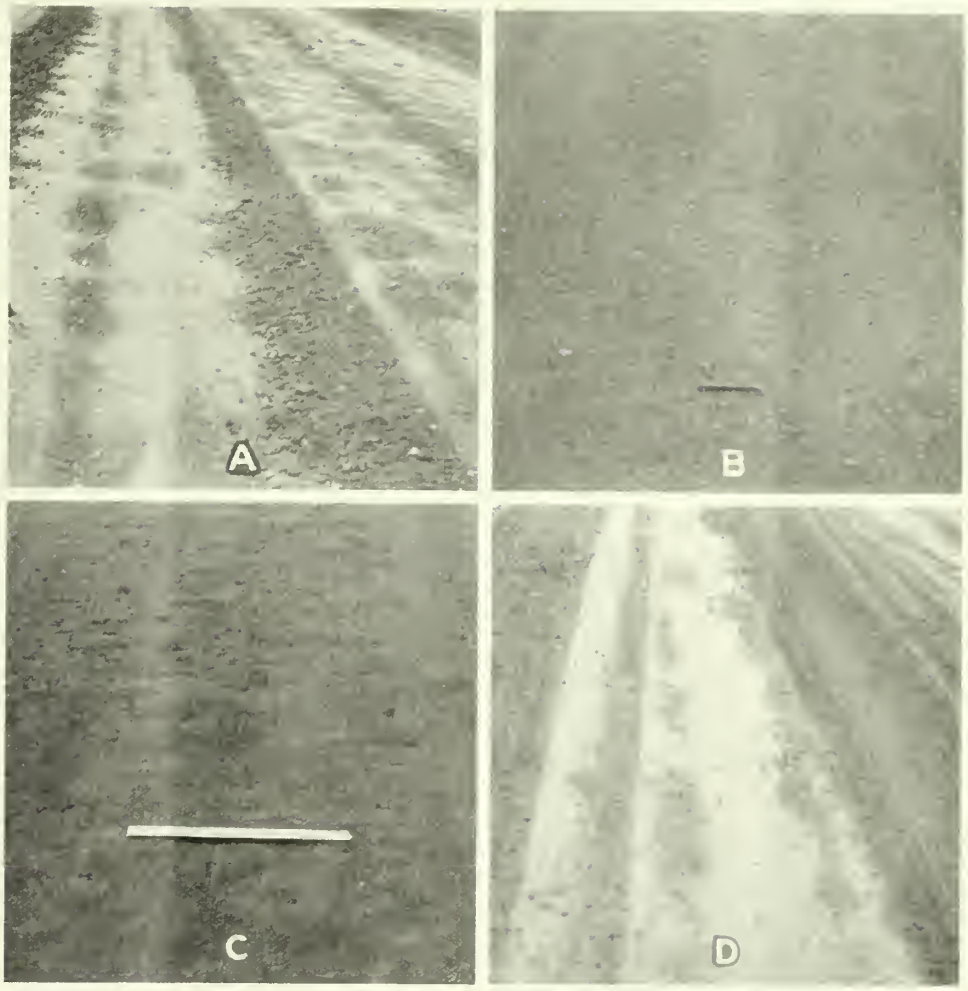


Figure 11.—Sections of Road Showing: A, Potholes Developed in Early Spring, B, Texture of Gravel Sections as Maintained Under Traffic, C, Sand Sections as Maintained Under Traffic Presenting a Highly Polished Surface Covered with Small Round Bumps, and D, the Slushy Condition of Surface During Heavy Rains After Blading Had Removed the Potholes. The Stabilized Material Beneath the Slush Is Firm and Compact.

Shrinkage cracks (fig. 10, *C*) occurred in the sections having a total thickness of 3 inches or less. No detrimental effects on the service behavior of the stabilized gravel were observed as a result of this condition.

Pitting (fig. 10, *D*) indicates the presence of abrupt surface cavities so small in size that, as a rule, they do not interfere with the riding qualities of the road. Potholes refer to cavities deep enough and large enough to cause excessive surface roughness (fig. 11, *A*).

Practically all of these defects occurred during dry summer weather. Areas, where the entire thickness of stabilized surfacing was displaced, were repaired by patching with a stabilized mixture of materials similar to those used in the original construction. The other defects were corrected with surface applications of calcium chloride and blading after rains. Following the blading of the moistened surface, no difference in condition was visible on the various sections.

A typical gravel section after traffic had compacted the bladed surface is shown in figure 11, *B*. The sand-clay sections (15 and 16) presented a surface highly polished and covered with small round bumps (fig. 11, *C*), which apparently resulted from the action of traffic on the fine-textured material containing some clay brought in from the shoulders during the blading operations. Further traffic abrasion caused this material to peel off and exposed the coarser textured sand-clay underneath.

Inspections during the winter disclosed that the surface was frozen solid and suffered very little wear. In the early spring, when most of the frost had left the ground and the surface was extremely wet as a result of melting snow and heavy rains, potholes developed over the entire project (fig. 11, *A*). Blading at this time filled the potholes and produced a smooth but slushy surface (fig. 11, *D*). The slushy material was splashed around a great deal by traffic but did not interfere in any way with the operation of vehicles. It is significant that this condition was confined to a thin layer on the surface. The stabilized material underneath was firm and compact and there was no rutting or breaking through to the subgrade.

The only condition which may be termed failure was where the entire thickness of stabilized mixture was displaced to the extent that the subgrade was exposed. This occurred on sections 9, 10, 11, 12, and 16. Failures were similar to those of section 1 prior to increasing the thickness from 1 inch to 4 inches.

Exposed subgrade was first observed on sections 10 and 9 (2 inches thick) within 2 weeks and 2 months, respectively, after completion of the road. The surfacing was worn off the center of the road for the entire

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TABLE 13.—*Results of condition surveys covering 2-year period*

Date of survey	CONDITION							
	1	2	3	4	5	6	7	8
9-24-36	Good	Good	Good	Good	Local rough areas	Local rough areas	Local rough areas.	Good
10-27-36	Good	Good	Good	Good	Good	Good	Good	Good
11-18-36	Good	Good	Deeply scaled from 629+50 to 632+50.	Shallow pitting and scaling.	Good	Very rough due to corrugations.	Good	Good
12-16-36	Good	Shallow raveling	About 2 inches worn off from 628 to 633.	Shallow raveling	Shallow raveling	Shallow raveling	Good	Good
3-11-37								
4-14-37								
4-29-37								
6-30-37								Shrinkage cracks.
7-23-37								
8-6-37								
8-24-37								
9-8-37								
4-13-38								
7-28-38								
CHARACTERISTICS								
Thickness (inches)	4	4	4	4	4	8	6	3
Liquid limit	23	21	23	24	18	25	22	23
Plasticity index	7	6	8	9	4	10	8	9
Subgrade	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil

length of section 10 (2 inches thick) after 1 year of service and section 16 (3 inches thick) after 2 years. At the end of the 2 years, the exposed areas on sections 9 (2 inches thick) and 11 (3 inches thick) were quite extensive while on section 12 (4 inches thick) they were small and scattered.

Section 12 differed from the other 4-inch sections in that the old bituminous mat was left in place on the subgrade when the stabilized surface was constructed. Owing to the difficulty in shaping the subgrade where the old bituminous mat was not removed, there may have been many places where the thickness was less than designed. This may also explain the greater wear on section 10 than on section 9.

The difference in behavior between sections 11 and 16 (3 inches thick) indicates that the stabilized sand offered less resistance to traffic abrasion than stabilized gravel.

BASE COURSE RECONSTRUCTED

The wear and tear on the stabilized base during the 2 years it was used as a road surface necessitated reconstruction of the entire base prior to bituminous surfacing in September 1938. New base-course material was added on some of the sections and the surface was merely reshaped on others. The materials were mixed in place on the road in contrast with the plant mixing used in the original construction.

Sections 9 and 10, originally constructed 2 inches thick, were rebuilt with thicknesses of 4 inches. Sufficient material was added to sections 2, 3, 4, 5, 8, 11, 12, and 16 to bring them to their original thickness of 3 or 4 inches. The work on these sections was performed by placing the proper amounts of sand or gravel and binder soil in windrows along the shoulder of the road and mixing with motor graders and a retread mixer consisting of a multiple-blade device which mixes and spreads material. The mixture was spread in thin layers, sprinkled by pressure distributors, and compacted by a pneumatic tire roller.

Sections 1, 6, 7, 13, 14, and 15 were reconditioned by blading to the proper cross section.

The average gradations of the materials combined on the road to produce the stabilized mixtures are shown in figure 12, *A*. The sand and gravel came from pit No. 588 and the binder soil was obtained from a pit located about 7.5 miles east of Ada. The binder soil was a heavy sandy loam which pulverized readily and

prior to placing bituminous surface from September 1936 to 1938

OF SECTIONS

9	10	11	12	13	14	15	16	Remarks
Good	Bituminous mat exposed locally.	Local rough areas.	Good	Local rough areas.	Good	Good	Good	Rough areas are corrugated.
Good	Patched	Good	Good	Good	Good	Good	Good	Bladed and reconditioned just prior to survey.
Subgrade exposed in local areas.	Patches destroyed	Slightly rough	Good	Good	Good	Good	Good	
Additional areas of exposed subgrade.	Additional areas of exposed mat.	Good	Good	Good	Good	Good	Good	
								No important changes in condition since December 16, 1936.
								Upper 2 inches of entire project softened as result of rains which occurred on Apr. 11, 12, and 13. Blading operations removed any differences in the condition of the various test sections. Subgrade was thawed to depth of 18 inches. No subgrade or base failures in evidence.
								Survey made during heavy rain. Portion of road had been bladed and was fairly smooth. Unbladed sections were rough and potholed. Surface of road was very slushy but there was no rutting or base failures. Stabilized material underneath the slush was firm and compact.
Exposed subgrade and shrinkage cracks.	Area of exposed mat increased.	Bituminous mat exposed locally. Shrinkage cracks.					Shrinkage cracks.	Potholes present in entire project in addition to condition noted on sections 8, 9, 10, 11, and 16.
								Following rains which occurred during week of July 12, the potholes were removed or partially filled by blading.
								Rains occurred on July 31 and August 1. Surface bladed August 2 and 3 filling potholes and producing smooth riding surface. By August 6 the material bladed into the potholes had been kicked out and the surface was again rough.
Area of exposed subgrade increased.	Bituminous mat exposed on entire section.	Area of exposed mat increased.	Bituminous mat exposed in local areas.	Shallow raveling and pitting.	Shallow raveling and pitting.	Shallow raveling and pitting.	Subgrade exposed in scattered areas.	Shallow raveling and pitting on sections 1 to 8. Weather quite dry since last survey.
								Heavy rains occurred Sept. 5. Blading on Sept. 6 removed irregularities and produced a smooth surface.
								Condition same as that on 8-24-37 except for additional areas of exposed subgrade and bituminous mat.
							Subgrade exposed on entire section.	Sections 1 to 15, inclusive, slightly more worn than shown by survey of 4-13-38. Final survey prior to placing bituminous surface.

OF SECTIONS

2	2	3	4	6	4	6	3
23	20	18	19	18	18	18	19
8	4	4	4	4	4	4	4
Soil	Bituminous mat	Bituminous mat.	Bituminous mat.	Bituminous mat.	Soil	Soil	Soil

was easily incorporated with the sand and gravel by road-mixing methods.

The thickness of the base course on each of the sections after reconstruction, together with the results of tests performed on the stabilized mixtures, are given in table 14. The range in gradations and average gradations are shown graphically in figure 12, *B*.

With respect to gradation, the materials in the different gravel sections were very similar. The mixtures in all sections had liquid limits less than 25 and, with the exception of sections 1, 6, and 7, had plasticity indexes not greater than 6. For the 14 gravel sections the average liquid limit was 18 and the average plasticity index was 4. The average density on the different sections just before placing the bituminous surface varied from 131 to 139 pounds per cubic foot.

Approximately 2 weeks after the base was reconstructed, a cut-back asphalt prime coat (MC–1) was applied at the rate of 0.20 to 0.25 gallon per square yard. This was followed by an application of 0.31 to 0.34 gallon per square yard of RC–2 cut-back asphalt and covered with 23 pounds per square yard of gravel having a maximum size of three-eighths inch. The surface was then broomed and rolled.

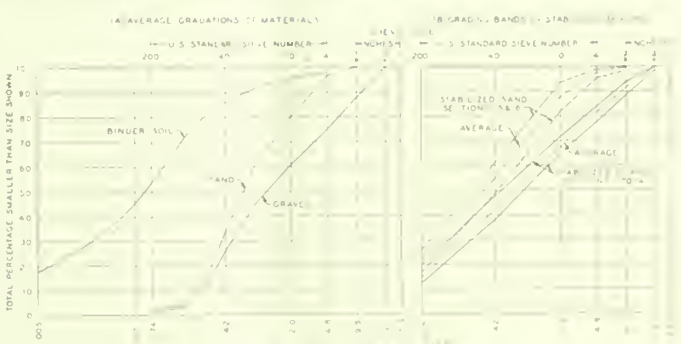

Figure 12.—Gradations of Materials and Mixtures, on the Reconstructed Base Course.

Seven days after the first gravel cover was applied the surface was given a second application of RC–2 material at the rate of 0.25 gallon per square yard and covered with sand at a rate of 12 pounds per square yard. This was followed by brooming and rolling. The resulting bituminous surface was approximately one-half inch thick.

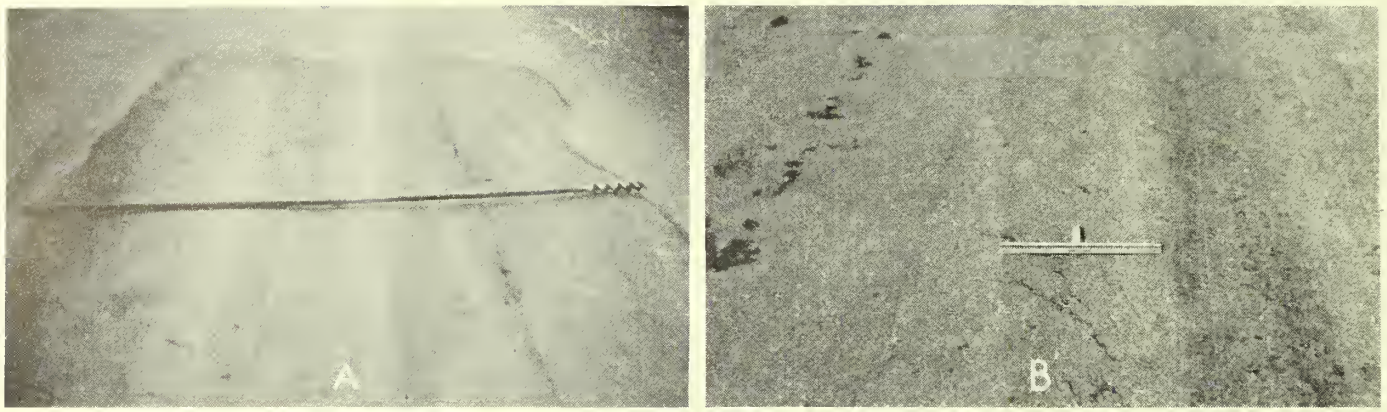

Figure 13.—A, Shoving in Bituminous Surface on Section 15, and B, Formation of Irregular Cracks During First Stages of Shoving.

Figure 14.—A Bituminous Surface That Failed After 1 Year of Service.

SHOVING OCCURRED IN BITUMINOUS SURFACE

Displacement or shoving of the bituminous surface developed quite extensively on section 15 during the summer of 1939. Figure 13, *A* shows the development of closely spaced irregular cracking or tearing resulting from destruction of bond within the mat on one side, and the accumulation, in the form of a ridge, of the shoved material on the other side. The formation of the irregular cracks during the first stages of shoving is illustrated in figure 13, *B*.

Table 14.—*Average thicknesses, gradation, and physical properties of stabilized mixtures of reconstructed base course*

Section number	Base thickness	Gradation						Ratio B/A	Liquid limit	Plasticity index
		Percentage passing sieve indicated								
		¾-inch	⅜-inch	No. 4	No. 10	No. 40	No. 200			
	Inches					(A)	(B)			
1	4.2	100	92	84	70	39	17	0.44	23	9
2	4.3	100	94	82	67	39	14	36	16	3
3	4.7	99	93	83	70	45	15	33	14	1
4	4.4	100	92	81	67	38	13	34	16	2
5	4.2	100	92	85	72	43	17	40	18	4
6	8.0	100	90	79	66	40	16	40	22	8
7	6.2	100	93	83	68	41	17	41	22	9
8	3.0	100	91	82	69	40	16	40	21	6
9	4.8	100	89	78	68	46	15	33	14	(1)
10	4.7	100	90	79	65	40	12	30	16	2
11	3.3	100	92	82	71	48	14	29	15	1
12	3.9	100	89	78	66	39	17	44	20	6
13	5.5	100	94	84	71	43	14	33	16	2
14	4.5	100	93	82	68	41	14	34	16	3
15	6.3	.....	100	99	94	61	26	43	19	4
16	4.8	.....	99	96	82	49	17	35	16	3

[1] Nonplastic value could not be obtained.

Investigation of the shoved areas in section 15 disclosed a plane of slippage at a depth of one-half inch or less below the bituminous mat. The lower portion, consisting of the original sand-clay base, was undisturbed and well consolidated. The upper portion of the base, displaced with the bituminous mat, consisted of a thin layer of sand-clay placed during the reconditioning of the base by blading back onto the road the material loosened by the abrasive action of traffic and accumulated on the shoulders. The plane of separation between the two layers was well defined and the surface of the lower portion of the base was smooth, indicating that the two layers were not bonded together and that the thin surface layer was sliding on the old base.

Displacement due to shoving, less in degree and extent than on section 15, was found on many of the other sections. Examination of these areas showed that there was a thin layer of loose material between the bituminous mat on top and the firm stabilized base below. On sections 9, 10, and 14 shoving occurred where an attempt had been made during reconstruction to bind and compact loose base-course material by the addition of some of the cut-back asphalt used in the surface.

All of the small areas where shoving developed were later ironed out but section 15 remained rough throughout the period of observations covered by this report.

CRACKING AND BREAKAGE INCREASED PROGRESSIVELY

After 1 year of service cracking and breakage of the bituminous surface began to develop on all sections except those having a base course 6 inches or more in thickness. Practically all failures were at the edges or within the outer 6 feet (fig. 14) where the base thickness was often found to be less than 3 inches.

In the spring of 1940, breakage had increased considerably over that of the previous year on those sections having bases less than 4 inches thick and failures were appearing in the 4-inch sections. Failures increased in area each year until the final survey was made in May 1942. The condition at that time is shown in figure 15.

All of the various types of failures mapped in figure 15 were accompanied by deformation. Slight deformation resulted in the typical alligator cracking shown in figure 16, *A*. As the deformation became greater, there was visible displacement of the base causing the surfacing to crack badly and peel off (fig. 16, *B*).

Breakage resulting from the cracking and deformation is illustrated in figures 16, *C* and *D*, and figure 17

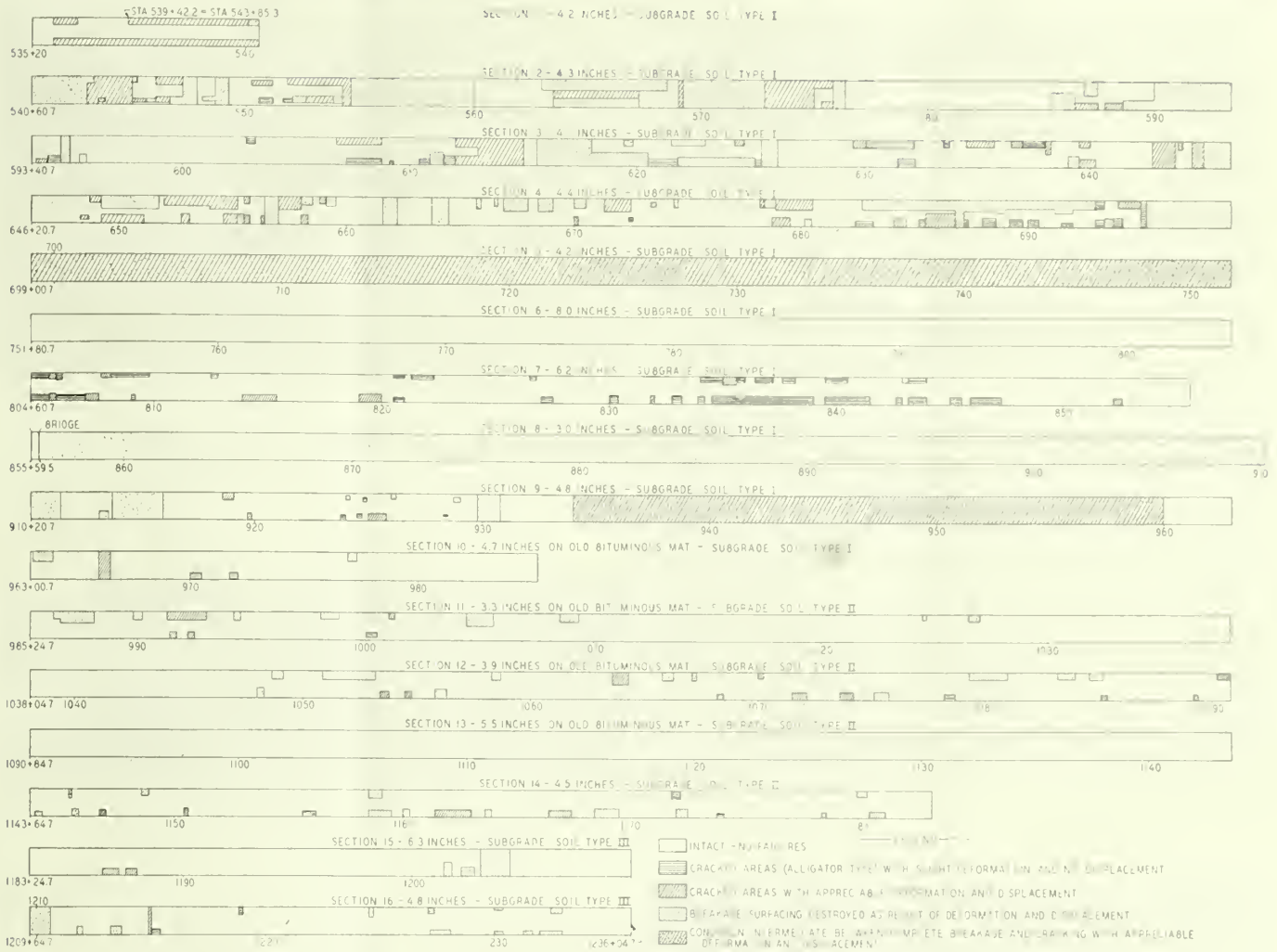
STA 539+42.2 = STA 543+85.3
SECTION 1 - 4.2 INCHES - SUBGRADE SOIL TYPE I
SECTION 2 - 4.3 INCHES - SUBGRADE SOIL TYPE I
SECTION 3 - 4 INCHES - SUBGRADE SOIL TYPE I
SECTION 4 - 4.4 INCHES - SUBGRADE SOIL TYPE I
SECTION 5 - 4.2 INCHES - SUBGRADE SOIL TYPE I
SECTION 6 - 8.0 INCHES - SUBGRADE SOIL TYPE I
SECTION 7 - 6.2 INCHES - SUBGRADE SOIL TYPE I
BRIDGE
SECTION 8 - 3.0 INCHES - SUBGRADE SOIL TYPE I
SECTION 9 - 4.8 INCHES - SUBGRADE SOIL TYPE I
SECTION 10 - 4.7 INCHES ON OLD BITUMINOUS MAT - SUBGRADE SOIL TYPE I
SECTION 11 - 3.3 INCHES ON OLD BITUMINOUS MAT - SUBGRADE SOIL TYPE II
SECTION 12 - 3.9 INCHES ON OLD BITUMINOUS MAT - SUBGRADE SOIL TYPE II
SECTION 13 - 5.5 INCHES ON OLD BITUMINOUS MAT - SUBGRADE SOIL TYPE II
SECTION 14 - 4.5 INCHES - SUBGRADE SOIL TYPE II
SECTION 15 - 6.3 INCHES - SUBGRADE SOIL TYPE III
SECTION 16 - 4.8 INCHES - SUBGRADE SOIL TYPE III
INTACT - NO FAILURES
CRACKED AREAS (ALLIGATOR TYPE) WITH SLIGHT DEFORMATION AND NO DISPLACEMENT
CRACKED AREAS WITH APPRECIABLE DEFORMATION AND DISPLACEMENT
BITUMINOUS SURFACING DESTROYED AS RESULT OF DEFORMATION AND DISPLACEMENT

Figure 15.—The Results of the Final Survey Showing the Condition of the Stabilized Base.

shows the condition of the road surface where no failures occurred.

PERFORMANCE RELATED TO BASE THICKNESS

Deterioration was most pronounced on sections 1 to 5 and sections 8 and 9. Section 8 (3-inch base), the first to show signs of failure, was completely destroyed after approximately 4 years of service. Sections 1, 2, 3, 4, 5, and 9, having base courses with average thicknesses ranging from 4.2 to 4.8 inches, were approaching destruction at the time of the final inspection in May 1942.

Section 6 (8-inch base) was entirely free of failures while slight deformation, accompanied by alligator cracking, was starting to appear in the outer 6 feet on both sides of section 7, which had an average base thickness of 6.2 inches.

Much less deterioration occurred in sections 10 to 13, inclusive, which had base courses ranging in thickness from 3.3 to 5.5 inches. On these sections, however, the old bituminous mat was not removed when the new base course was constructed. On the contrary, repairs were made to true up the surface as much as practicable. The old mat was from 1½ to 2 inches thick and apparently was equivalent in traffic service to at least an equal thickness of stabilized base.

Section 13, having a base course 5.5-inches thick plus approximately 2 inches of the underlying bituminous mat, showed no signs of cracking or deformation and was just as sound in every respect as section 6, which had an 8-inch base placed directly on the clay subgrade. The benefit derived from the added thickness supplied by the old bituminous mat is further illustrated by the condition of section 9 compared with section 10 and of section 8 compared with section 11.

Although failures occurred in section 14, it was in much better condition than sections 1, 2, 3, 4, 5, and 9, which had approximately the same thicknesses of base. The densities as well as the physical properties of the subgrade soil in section 14 indicate a more stable subgrade, and consequently better support. This may account for the difference in behavior.

The condition of sections 15 and 16 as compared to that of other sections demonstrated conclusively that, for equal thicknesses, the performance of the stabilized sand base course was equal to that of the stabilized gravel base.

While the higher subgrade densities and lighter texture of the soil in sections 11 to 16 appear to have had some bearing on the behavior of the test sections, it is believed that these properties were of minor importance compared to the influence of base thickness.

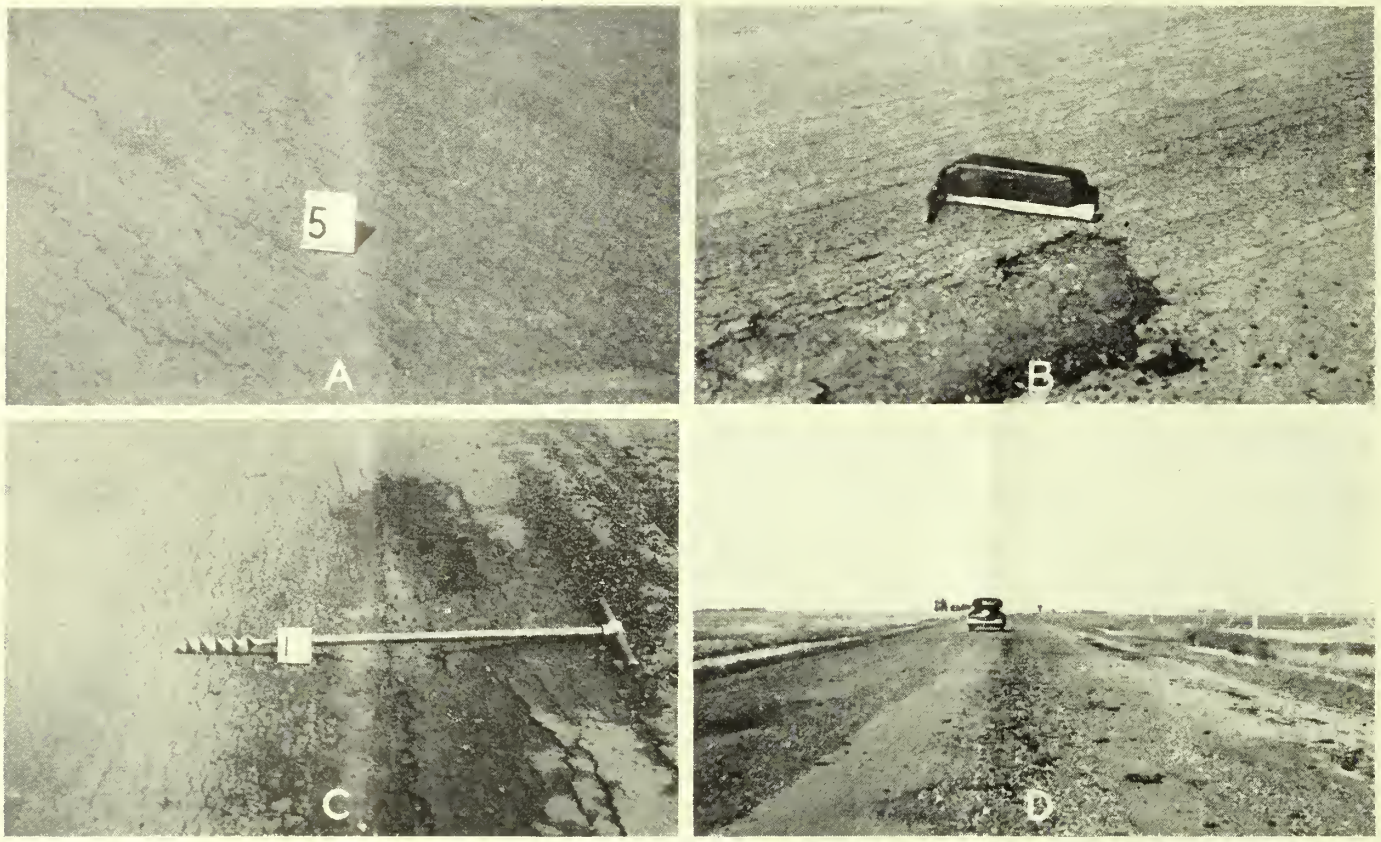

Figure 16.—Deformation of Surface by Cracking: A, Alligator Type Cracking With Slight Deformation, B, Appreciable Deformation and Displacement Causing the Surface To Crack Badly and To Peel Off, C, Breakage, and D, Surfacing on Section 8 Completely Destroyed.

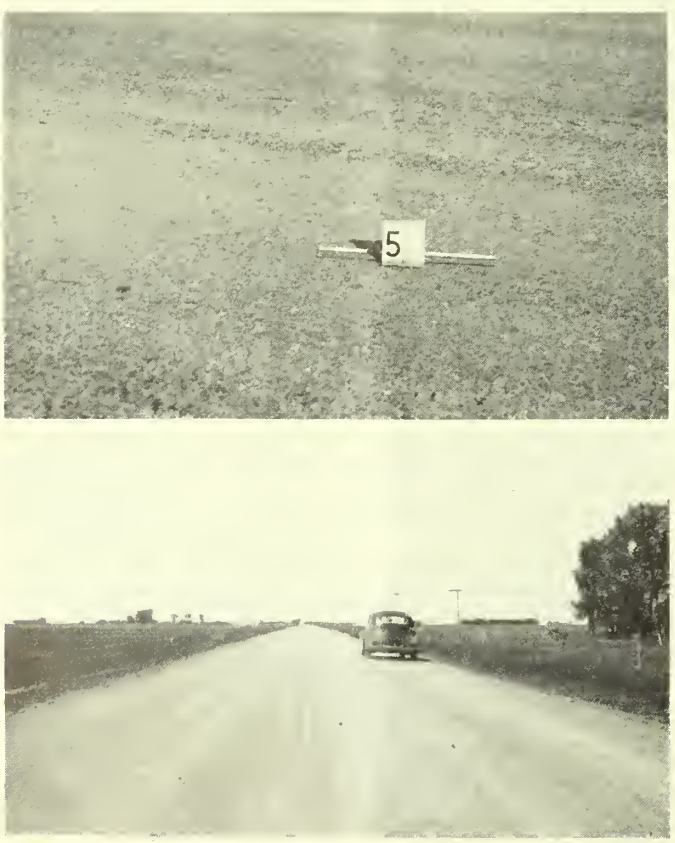

Figure 17.—Surfaces Free of Failures During 4 Years of Service.

SUMMARY AND CONCLUSIONS

With respect to materials and methods of construction, the foregoing discussion may be summarized as follows:

Heavy clay soils may be combined with granular material to produce stabilized mixtures of uniform quality by means of the mixing plant described in this report without any previous preparation such as drying and pulverizing generally required in ordinary road-mix methods of construction.

Neither the origin nor the chemical composition of the binder soils used in this experiment had any influence on the performance of the stabilized mixtures. For equal proportions of binder soil and granular material, the liquid limits and plasticity indexes of the resulting mixtures varied in accordance with liquid limits and plasticity indexes of the binder soils regardless of their source, organic content, or lime content.

A definite increase in compaction (approximately 4 percent) under the action of traffic was observed during the first month following construction with practically no increase thereafter.

Calcium chloride in the amount used integrally on this work (0.45 percent by weight or approximately 1 pound per square yard for a 2-inch layer) was not sufficient to maintain the surface in a moist condition during the hot, dry weather characteristic of the construction period. Additional applications of calcium chloride (about 0.5 pound per square yard) after light rains or sprinkling helped considerably towards maintaining a smooth surface.

(*Continued on p. 54.*)

A STUDY OF THE PAT TEST FOR DETERMINING ALKALI-REACTIVE AGGREGATES

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by D. G. RUNNER, Materials Engineer

DURING THE PAST FEW YEARS, the marked expansion that has been observed in certain concrete structures, particularly in the West, has raised a question as to the possibility of a reaction between the alkalies in the cement and certain constituents in the aggregates. The interest in this problem is indicated by the amount of literature [1] on the subject that has been developed within a relatively short time and is evidence that considerable time and effort have been spent in attempting to understand something of the nature of this reaction.

Recent experiences in California (*13*) and Washington (*15*) have focused attention upon a chemical reaction between certain aggregates, or ingredients in aggregates, and the alkali content of the cements as one of the major factors in the deterioration of concrete. In many instances this reaction has caused a volume change of sufficient magnitude to induce serious failure of the concrete.

One fact brought out by investigators is that opal and volcanic glass react with cements high in total alkali content.

The usual and more or less standard practice in identifying alkali-reactive aggregates has been to measure length changes of small mortar bars of about a 1:2 mix which are placed in moist storage for an indefinite period. This practice precludes the use of the procedure as a rapid laboratory test and, with this thought in mind, the California Division of Highways proposed a so-called pat test which was found to shorten considerably the time necessary to determine whether or not a given aggregate is reactive with alkalies in the cement (*14*).

In view of the obvious need for a test of this nature it seemed desirable to conduct additional tests to study further the nature of the reaction as well as to determine the extent of application of the procedure. For this reason several short series of tests involving a rather wide variety of aggregates and some modifications of the original procedure have been made by the Public Roads Administration. The observations from these tests have been assembled in this report with the thought that the trends revealed may be of some value to engineers interested in this subject.

It should be emphasized that these tests were not conducted as parts of a coordinated program planned in advance. They represent for the most part tests made from time to time with materials on hand in the laboratory. This accounts for the obvious gaps in the data, especially the lack of a more complete coverage of possible variations in alkali contents of cements.

METHOD OF MAKING THE PAT TEST

In making this test on coarse aggregates, pats of neat cement about three-fourths inch thick and 4½ inches in diameter are prepared and the broken stone or gravel fragments are embedded in the top of the fresh paste. In the investigation of sand aggregate a 1:2 mortar is used. After molding, the pats are placed in the moist closet for at least 24 hours, after which the surface of the pat is ground smooth to expose the aggregate. The grinding is done on a metal lap using water and No. 220 carborundum grit as a grinding agent. The pat is then washed clean and stored for 4 days at normal temperature in an airtight container in a solution of one part one-half normal solution NaOH, one part one-half normal solution KOH, and two parts saturated limewater.

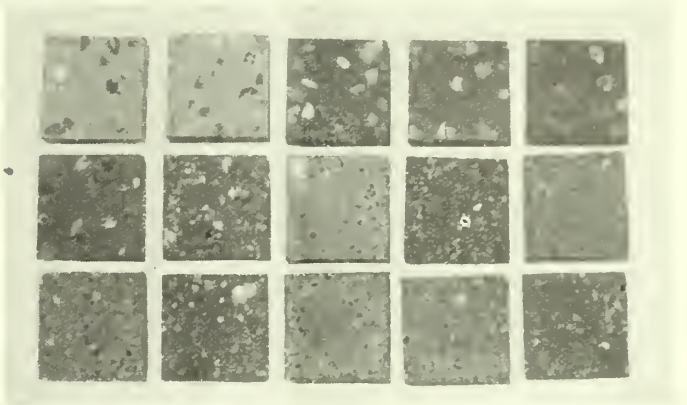

Figure 1.—View of Laboratory Set-Up for Making Pat Tests.

In the tests reported herein, the only major deviations from the California method consisted in reducing the size of the pats from 4½ inches in diameter to 3-inch squares, and in placing the broken stone and gravel fragments in the bottom of the mold and covering them with the cement paste. This procedure made certain that each fragment would be at or near the surface of the pat upon removal from the mold, and in addition made the molding operations a little easier. The Public Roads Administration method of storing the pats is illustrated in figure 1. A reactive aggregate produces a whitish reaction product, which appears on and near the exposed aggregate particle. This product is assumed to result from the activity of the alkalies in the cement and certain ingredients in the aggregate. A test period of 5 days has been suggested, but in many instances periods greater than this were necessary for the reaction product to appear. The physical appearance of an unusually reactive substance is shown in figure 2.

TESTS WITH OPAL

According to Stanton (*14*) the concrete failures in California reported earlier were caused by the reaction between opaline silica, or impure limestone, and the alkalies in the cement. Although other materials have been found to be reactive, Stanton states that opal has been proven definitely to be a highly reactive mineral

[1] Italic numbers in parentheses refer to bibliography at end of article.

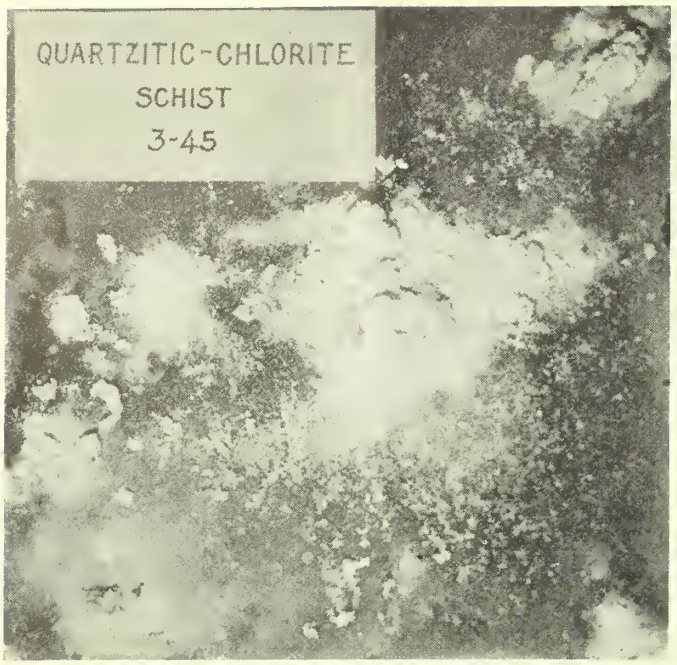

Figure 2.—Reaction Product Formed With Quartzitic-Chlorite Schist After 63 Days in Caustic Solution. Cement Contained an Average of 0.92 Percent Total Alkalies.

and with one exception is an essential part of all California rocks identified as reactive.

For the tests reported herein, supplies of semiprecious and wood opal were purchased. As a matter of general interest the properties of this mineral as listed by Dana and Ford (7) are:

Mineral class:	Oxide
Composition:	$SiO_2\ nH_2O$, with percentage of water varying from 2 to 13 percent
Hardness:	5.5 to 6.5
Specific gravity:	1.9 to 2.3; when pure 2.1 to 2.2
Luster:	Vitreous
Color:	White, yellow, red, brown, green, gray, blue
Fracture:	Conchoidal
Crystal form:	Amorphous (colloidal)
Varieties:	Precious, fire-opal, wood-opal, hyalite, cacholong, milk-opal, hydrophane, * * *
Chemical properties:	* * * Soluble in hydrofluoric acid somewhat more readily than quartz; also soluble in caustic alkalies, but more readily in some varieties than in others.
Observations:	Opal is a non-crystalline, colloidal substance belonging to the group of the mineral-gels. It has been deposited at low temperatures from silica-bearing waters. It occurs in connection with many rock types, igneous, sedimentary, and metamorphic. It is found filling seams and fissures of igneous rocks * * * It occurs in some mineral veins and is deposited from many hot or warm springs. It is further formed during the weathering and alteration of many rocks.

The data developed in the investigation of the reaction between opal and cement are shown in table 1. Two variations of opal were tested for reactiveness under several different conditions. Samples of semiprecious opal placed in pats of high alkali cement produced reaction within 2 days, although in one instance a reaction product did not become evident until the 18th day. It was thought that the water content of the opal might explain its reactive characteristics. Three samples were heated in a test tube until the water was given off, as evidenced by change in color of the mineral, and the amount of moisture collected on the side of the test tube. Pats containing this dehydrated opal were stored in the caustic solution and moderate to heavy reaction was noted in 5 to 18 days. As far as could be determined, the reactive product from the dehydrated opal was in no way different from the hydrated or regular opal. Wood opal, tested under hydrated and dehydrated conditions, in general, produced similar results.

In further tests with wood opal three pats were made, each differing from the others in alkali content of the cement. Instead of regular tap water, the cement was mixed with one-half normal caustic solution such as was used in the soaking bath. After 220 days none of the opal showed any visible signs of reacting with any of the cements. The reason for this behavior is not completely understood at this time. However, it is probable that the caustic solution used as mixing liquid had a neutralizing effect. Possibly reaction is delayed and will commence at some future time.

The thought has occurred from time to time that the aggregates used in the pat test may react with the alkalies in the caustic solution instead of the alkalies in the cement. To investigate this possibility wood opal was used in three pats; one containing high alkali cement, the second a medium alkali cement, and the third a low alkali cement. All three pats were then immersed in distilled water. In 12 days the opal in the high alkali cement pat showed signs of reaction, and in 10 days the medium alkali cement and opal began to react. There were no signs of reaction in the low alkali cement pat at the end of 185 days.

In an effort to retard the reaction between opal and high alkali cement, a pat containing these ingredients was placed in a caustic solution and kept at a tempera-

Table 1.—*Results of tests with various samples of opal*

Varieties and sample	Alkalies in cement		Number of days in caustic solution to produce reaction indicated			
	Na_2O	K_2O	None	Slight	Moderate	Heavy
Semiprecious:	*Percent*	*Percent*				
A	1.13	0.02			2	4
B	1.13	.02			18	30
C	1.13	.02			5	
D	1.13	.02				5
Semiprecious, dehydrated:						
A	1.13	.02			18	30
B	1.13	.02			5	
C	1.13	.02				5
Wood						
A	1.13	.02		1		2
B	1.13	.02		1		2
C	1.13	.02			3	
D	1.13	.02				4
Wood, dehydrated:						
A	1.13	.02				4
Wood [1]:						
A	1.13	.02	220			
B	.13	.22	220			
C	.10	.12	220			
Wood [2]:						
A	1.13	.02		12	240	
B	([3])	([3])		10	185	
C	.10	.12	185			
Wood [4]:						
A	1.13	.02	([5])			
Hyalite:						
A	1.13	.02	150			
B	.13	.22	150			

[1] One-half normal caustic solution (NaOH+KOH) used as "mixing water" in place of tap water.
[2] Pats stored in distilled water instead of caustic solution.
[3] The sum of the alkalies is 0.68.
[4] Pat stored in caustic solution at 45° C. for 30 days.
[5] Opal completely dissolved at end of 30 days (see fig. 3).

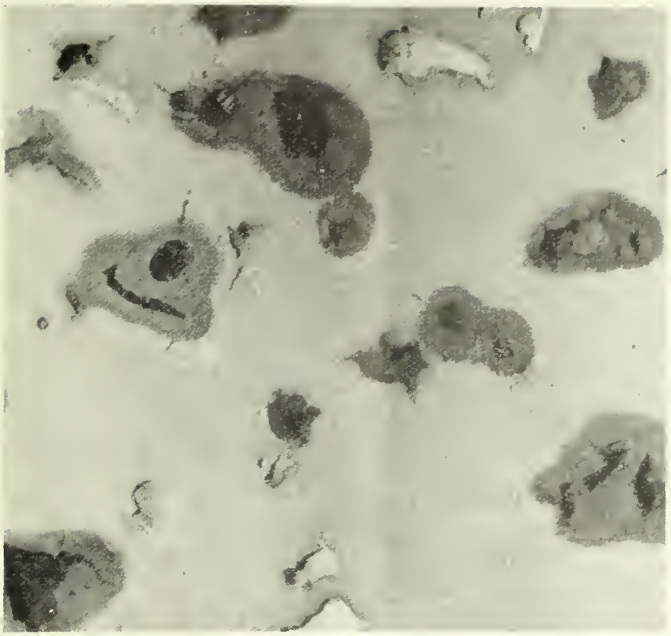

Figure 3.—Dissolving Effect of Caustic Solution Upon Wood Opal After 30 Days at 45° C. Cement Contained 1.13 Percent Na_2O and 0.02 Percent K_2O. Dark Areas Around Holes Are Moisture Remaining After Pat was Washed in Tap Water.

ture of 45° C. (113° F.) for 30 days. At the end of this period there was no typical reaction product such as usually was found, but the wood opal had been dissolved, so that upon washing the pat nothing remained except a mold or cast where the opal had been embedded. This condition is shown in figure 3. The dark spots around the cavities are moisture remaining from the washing operation.

Hyalite, a whitish translucent variety of opal, developed no signs of reaction after 150 days' storage in caustic solution.

To further investigate the reaction between opal and alkalies, a pat was made using a cement with a total alkali content of 1.15 percent and containing several pieces of wood opal. After curing for 24 hours, and after being removed from the mold, the pat was placed on a shelf in the moist closet. The temperature and humidity were 70° F. and 95 percent, respectively. In about 1 month the reaction product began to form. At times there was a jellylike globule at the junction point of the opal and the cement. Eventually this globule would dry, leaving a white precipitate in its place. The condition of this pat after 8 months storage in the moist closet is shown in figure 4. From all indications, it must be concluded that the opal reacts with certain ingredients in the cement, probably the alkalies. The results of these tests are not tabulated.

USE OF CEMENTS WITH DIFFERENT TOTAL ALKALI CONTENT

Considerable research has been carried out in connection with cements containing relatively high alkali content, and aggregates known to be reactive. The question arose regarding the reaction of cements of low alkali content. To investigate this problem several aggregates, all of which were suspected of being reactive, were made into pats and then immersed in caustic solution. The total alkali content of the cements used ranged from 0.19 to 1.15 percent. The results obtained with these cements are shown in table 2.

Figure 4.—Reaction Formed on Pat Prepared With Wood Opal and Cement Containing 1.13 Percent Na_2O and 0.02 Percent K_2O, After 8 Months' Storage in Moist Closet at a Temperature of 70° F. and a Humidity of 95 Percent.

Table 2.—*Results of tests using cements of high, medium, and low alkali content*

Aggregate	Alkalies in cement		Number of days in caustic solution to produce reaction indicated		
	Na_2O	K_2O	None	Slight	Heavy
	Percent	*Percent*			
Feldspar basalt	(1)	(1)		35	
	(2)	(2)		35	
Olivine basalt	(1)	(1)	63		
	(2)	(2)	63		
Tuffaceous rhyolite	(1)	(1)		35	63
	(2)	(2)		35	63
Sand, Burnt River [3]	1.13	.02		3	
	(4)	(4)		3	
	.10	.12		3	
Gravel, Burnt River [3]	1.13	.02		3	13
	(4)	(4)		3	13
	.10	.12		3	13
Sand, Toutle River [5]	1.13	.02		39	
	(4)	(4)		39	
	.10	.12		39	
Gravel, Toutle River [5]	1.13	.02		16	
	(4)	(4)		38	
	.10	.12		16	

[1] The sum of the percentages of alkalies is 0.92.

[2] The sum of the percentages of alkalies is 0.19.

[3] Contains rhyolite, granite, quartzite, basalt, quartz plus minor quantities of other rock types.

[4] The sum of the alkalies is 0.68.

[5] Contains basalt, andesite, rhyolite, felsite plus minor quantities of other rock types.

It was necessary in most cases to keep the specimens in solution longer than 5 days in order to develop reaction products. It is interesting to note that reaction was obtained with low alkali cement as well as with high alkali cement. Photographs of the reaction obtained by both low and high alkali cements with tuffaceous rhyolite are shown in figure 5.

In the case of sand and gravel from the Burnt River in Oregon and the Toutle River in Washington pats

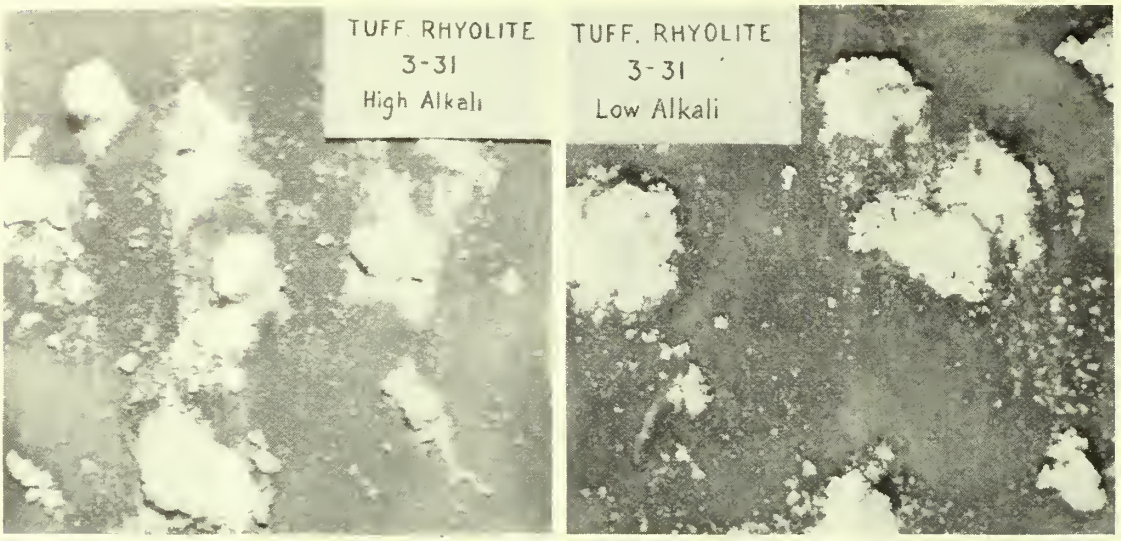

Figure 5.—Reaction Product Formed Using a High Alkali and a Low Alkali Cement with Tuffaceous Rhyolite, After 63 Days' Storage in Caustic Solution. The High Alkali Cement Contained an Average of 0.92 Percent Total Alkalies and the Low, 0.19 Percent.

were made using high, medium, and low alkali cements. In every instance reaction took place with each cement and aggregate. The aggregate from the Burnt River developed slight reaction at 3 days, while it took considerably longer for the aggregate from the Toutle River to react. In the latter case, the reaction was slight but, nevertheless, it was indicative that some reactivity could be expected when this aggregate and a cement containing at least 0.22 percent total alkali were used. Petrographic analyses of Burnt River and Toutle River aggregates show a preponderance of the igneous rock types, among which are basalt, rhyolite, felsite, and granite. According to available information these materials have a doubtful service record. The test results reported herein indicate that they will react unfavorably with cements low in total alkalies as well as with those containing relatively large amounts of total alkalies.

TESTS ON VARIOUS MINERALS

In an effort to identify the minerals reacting with high alkali cements, a number of minerals ranging widely in chemical composition were selected for tests. These minerals consisted chiefly of the feldspars, zeolites, and silicates with one fluorite, a carbonate, and a silicate containing beryllium. The complete list of minerals and the results obtained are shown in table 3. The feldspars and silicic minerals were included because of their suspected reaction, and connection with distress exhibited by concrete in which aggregate from the Platte River areas was used. The zeolites and remaining minerals were added in the hope that they might furnish some information regarding the reaction between minerals of certain chemical composition and the alkali content of cement. All of the minerals shown in table 3 were tested in pats containing high alkali cement which were in storage in the solution from 53 to 74 days. The reaction in every case was negative.

TESTS ON ROCK OF VARIOUS TYPES

In a further study of the pat test, it was decided to test various types of igneous, sedimentary, and metamorphic rock. With few exceptions, all the pats were made up with cement containing 0.92 percent total alkalies. The results obtained with these various rocks are shown in table 4.

Referring to table 4, it will be noted that only eight rocks developed any reaction products, and the length of time required to develop the reaction ranged from 3 to 65 days. Of the rocks in the igneous group that were reactive, two could be classed as acid type (obsidian and dacite), one of intermediate type (trachyandesite), and one of basic type (feldspar basalt), all based upon the probable silica content of the rock. It took 9 days for any reaction to develop in the case of the igneous rocks against 3 days for the metamorphic and 4 days for the sedimentary rocks. This difference in time may be due to the rock texture as well as to the mineral content of the rocks. Two of the igneous rocks classed as reactive have been identified as "altered," that is, some of the minerals in the rocks have been changed from their original composition, either by weathering or other means. Although there is nothing definitely conclusive about this feature, indications are present and it might be well to investigate completely before use, any aggregate that has been subject to weathering, especially if a high alkali cement is to be used with it. Figure 6 illustrates the reaction obtained with altered trachyandesite.

Referring to table 4, the materials identified as opaline chert and siliceous-magnesian limestone proved

Table 3.—*Results of tests on various minerals*

Mineral	Chemical composition [1]	Alkalies in cement		Number of days in caustic solution	Reaction produced
		Na_2O	K_2O		
		Percent	*Percent*		
Microcline	$K_2O.Al_2O_3.6SiO_2$	1.13	0.02	54	None.
Anorthite	$CaO.Al_2O_3.2SiO_2$	1.13	.02	54	Do.
Scolecite	$CaO.Al_2O_3.3SiO_2.3H_2O$	1.13	.02	54	Do.
Prehnite	$H_2Ca_2Al_2(SiO_4)_3$	1.13	.02	54	Do.
Calcite	$CaCO_3$	1.13	.02	54	Do.
Quartz	SiO_2	1.13	.02	53	Do.
Diatomite	Essentially SiO_2	1.13	.02	53	Do.
Chert (white)	SiO_2	1.13	.02	54	Do.
Chert (brown)	SiO_2	1.13	.02	54	Do.
Fluorite	CaF_2	1.13	.02	53	Do.
Beryl	$3BeO.Al_2O_3.6SiO_2$	1.13	.02	74	Do.

[1] The chemical compositions of the minerals are those given by Dana and Ford (7).

TABLE 1.—*Results of tests on igneous, sedimentary, and metamorphic rocks*

Rock type	Classification	Source	Total alkalies in cement	Number of days in caustic solution to produce reaction indicated			
				None	Slight	Moderate	Heavy
			Percent				
Obsidian	Igneous	Oregon	[1] 0.92			15	
Do	do	do	.92	65			
Vitreous basalt	do	do	.92	65			
Feldspar basalt	do	do	.92	65			
Trachyte	do	do	.92	65			
Altered trachyandesite	do	do	.92		9		65
Pumice	do	do	.92	65			
Do	do	do	.92	65			
Altered dacite	do	do	.92		35		41
Altered diabase	do	Washington	.92	65			
Granite	do	Arizona	.92	65			
Augite syenite	do	do	.92	65			
Olivine basalt	do	California	.92	65			
Tuffaceous rhyolite	do	Colorado	.92	65			
Feldspar basalt	do	New Mexico	.92		64		
Basalt	do	Wyoming	.92	65			
Do	do	do	.92	65			
Obsidian	do	do	[2] 1.15	65			
Do	do	do	[3] .22	65			
Gabbro	do	Virginia	1.15	64			
Opaline chert	Sedimentary	California	[1] .92		4		6
Siliceous-magnesian limestone.	do	do	.92		5		8
Porous sandstone	do	Oregon	.92		4		6
Argillaceous sandstone	do	Colorado	.92	65			
Conglomeratic sandstone	do	New Mexico	.92	65			
Pyroxene quartzite	Metamorphic	Colorado	[1] .92	65			
Quartzitic-chlorite schist	do	South Dakota	.92		3		5
Micaceous quartzite	do	do	.92	65			

[1] Three cements were blended for these tests. Average total alkali content, 0.92 percent. Range from 0.85 to 1.06 percent.
[2] Na_2O=1.13 percent, K_2O=0.02 percent.
[3] Na_2O=0.10 percent, K_2O=0.12 percent.

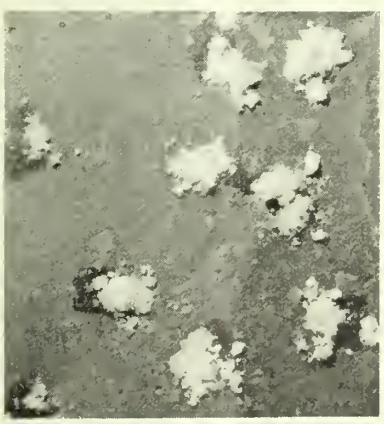

Figure 6.—REACTION OBTAINED IN 65 DAYS WITH CEMENT CONTAINING 0.92 PERCENT TOTAL ALKALIES AND ALTERED TRACHYANDESITE, AN IGNEOUS ROCK TYPE.

to be very reactive, and the data of these tests correlate with the results obtained by the California Division of Highways. Photographs of the reaction developed by these two aggregates are shown in figure 7.

Porous sandstone developed a slight reaction in 4 days and a much heavier reaction in 6 days. The extent of the reaction after about 9 weeks in the caustic solution is shown in figure 8.

The only metamorphic rock, of the three tested, to develop any reaction was a quartzitic-chlorite schist. Here, the reaction was noticeable within a period of 3 days. Figure 2 shows the reaction at the end of 63 days.

Figure 7.—REACTION OBTAINED USING OPALINE CHERT AND SILICEOUS-MAGNESIAN LIMESTONE WITH CEMENT CONTAINING 0.92 PERCENT TOTAL ALKALIES.

RESULTS OF TESTS ON SAND AND GRAVEL

The results obtained by the pat test method with a number of commercial aggregates, mostly sands and gravels from different areas of the United States, are shown in table 5. Because of the interest evidenced in the disintegration of certain concrete roads in Nebraska, a number of aggregates from this region are included in the group shown in table 5. The table shows that only slight action was developed in four of the samples from Nebraska and the number of days necessary to produce this reaction ranged from 35 to 103. The evidence of the reaction for one aggregate, which is typical, is shown in figure 9. The only other material to develop reaction products, was the cherty sand from Tennessee. Here again the reaction was classified as slight and 63 days was necessary to produce it. There

TABLE 5.—*Results of tests of various types of sands and gravels*

Kind and mineralogical type of aggregate	Source	Alkalies in cement		Days in caustic solution to produce—	
		Na_2O	K_2O	No reaction	Slight reaction
		Percent	*Percent*		
Gravel:					
Siliceous	Scottsbluff, Nebr	(1)	(1)		103
Do	North Platte, Nebr	(1)	(1)		36
Do	Lexington, Nebr	(1)	(1)	90	
Do	Kearney, Nebr	(1)	(1)	90	
Do	Grand Island, Nebr	(1)	(1)		35
Do	Central City, Nebr	(1)	(1)	90	
Do	Columbus, Nebr	(1)	(1)	90	
Do	South Bend, Nebr	(1)	(1)	90	
Do	Oreapolis, Nebr	(1)	(1)		38
Do	Louisville, Nebr	(1)	(1)	90	
Sand:					
Siliceous	Platte River, Nebr	0.36	0.56	63	
Do	do	.26	.67	63	
Do	do	.26	.47	63	
Do	do	.19	.43	63	
Cherty	Estill Springs, Tenn	.36	.56		63
Do	do	.26	.67		63
Do	do	.26	.47		63
Do	do	.19	.43		63
Quartzitic	Long Island, N. Y	.36	.56	63	
Do	do	.26	.67	63	
Do	do	.26	.47	63	
Do	do	.19	.43	63	
Stone sand:					
Calcareous	Chicago, Ill	.36	.56	63	
Do	do	.26	.67	63	
Do	do	.26	.47	63	
Do	do	.19	.43	63	
Sand:					
Siliceous	Alexander, N. Y	1.13	.02	63	
Do	Albany, N. Y	1.13	.02	63	
Do	Alfred, N. Y	1.13	.02	63	
Do	Springville, N. Y	1.13	.02	63	

[1] The sum of the alkalies is 0.92 percent.

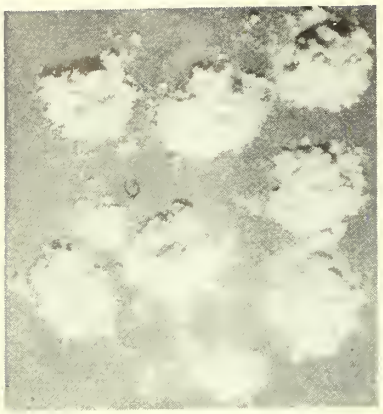

Figure 8.—Extent of Reaction Product Developed Between a Porous Sandstone and Cement Containing an Average of 0.92 Percent Total Alkalies, After 9 Weeks' Storage in Caustic Solution.

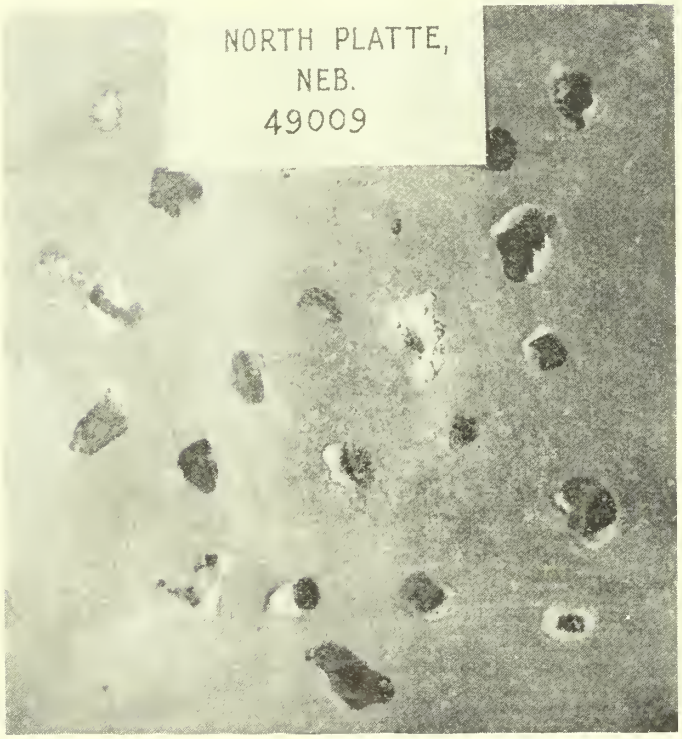

Figure 9.—Reaction After 36 Days in Aggregate From North Platte, Nebr. Cement Contained an Average of 0.92 Percent Total Alkalies.

is a possibility that the chert represented by this sample contains opaline silica, or some other form of silica may have been the active ingredient. The total alkali contents of the cements used in testing this aggregate ranged from 0.69 to 0.93 percent, so that they could be called medium to high alkali cements.

The stone sand from Chicago and the aggregates from New York State did not develop any reaction. The durability of these aggregates when used in concrete has been questioned. The fact that they developed no reaction products in the pat test, does not necessarily mean that they are free from suspicion.

INVESTIGATION OF AGGREGATES FROM PACIFIC NORTHWEST

Because of the disintegration of concrete in Washington and Oregon, and the widespread interest in the

Table 6.—*Chemical analyses of cements used with aggregates from the Pacific northwest arranged in decreasing total alkali content*

Sample	SiO_2	$Al_2O_3+Fe_2O_3$	CaO	MgO	SO_3	Ignition loss	Insoluble residue	Na_2O [1]	K_2O [1]	Total alkalies	Specific surface
											Cm.² per gm.
A	21.98	8.06	62.58	1.82	1.76	2.89	0.19	0.61	0.42	1.03	1,750
B	19.58	10.02	63.52	2.60	1.26	2.24	14	58	39	97	1,680
C	20.68	8.00	63.34	2.38	1.76	3.00	23	21	75	96	1,870
D	21.62	7.90	65.32	1.39	1.64	1.28	13	18	64	82	1,860
E	22.22	8.32	62.24	2.97	1.26	2.22	16	50	28	78	1,920
F	21.42	8.70	64.22	1.96	1.25	1.51	14	52	22	74	1,600
G	22.18	7.62	64.02	1.88	1.46	2.56	15	49	23	72	1,590
H	22.76	6.84	66.08	0.78	1.93	1.33	15	22	45	67	1,790
I	20.98	9.50	64.56	2.14	1.53	1.17	06	46	14	60	1,800
J	24.50	4.94	66.26	1.18	1.56	1.26	09	25	27	52	1,930
K	23.46	7.00	64.78	1.64	1.56	1.01	20	13	22	35	1,680
L	21.98	7.44	65.12	2.14	1.78	1.50	23	11	16	27	1,780

[1] A. S. T. M. Method C 114-40 used in determining these oxides.

reaction between certain aggregates and cements it was decided to include aggregates and various cements from this immediate locality. The aggregates tested were produced mainly in Washington and Oregon and have been used for a number of years in pavement and bridge construction. With one exception, the cements used with the aggregates in this part of the investigation were produced in the northwest. The chemical analyses of these 12 cements are given in table 6. The specific surface fineness ranged from 1,590 to 1,930 sq. cm. per gm. The results obtained in this part of the investigation on reactive aggregates are shown in table 7. Five of the aggregates gave negative results with all 12 cements. Sand and gravel from the Cowlitz River in Washington, sand from Grandview, Wash., and both sand and gravel from the Toutle River in Washington, produced little or no reaction. However, table 2 shows that aggregate from the Toutle River with cement containing 1.15 percent total alkalies produced reaction in from 16 to 39 days. This aggregate was also used to develop the information given in table 7, but was used at a different time.

A possible reason for some of the negative results indicated in table 7 for certain of the cements may have been the small amount of the reactive material in the entire sample as submitted to the laboratory.

The greatest amount of reaction seems to have taken place in the aggregate from Pocatello, Idaho; the White River in Washington; Dixie School, Oreg.; and the Burnt River in Oregon. Four of these aggregates developed a reaction with all 12 cements while in the combinations of the other 3 aggregates with the 12 cements there were only 9 which failed to develop a reaction. It will be recalled that the total alkalies of the cements ranged from 0.27 to 1.03 percent (see table 6). This indicates that certain aggregate types will react with low alkali cements as well as with high alkali cements. This is shown clearly in figure 10, where the extent of the reaction is very marked for all 12 cements. Petrographic analyses indicate that in general the aggregates showing reaction contained one of the following minerals: Quartz, rhyolite, basalt, felsite, granite, limestone, and sandstone. In view of the wide variation in the mineral content, it is a very difficult matter to discredit any one type of aggregate. Much research remains to be done with minerals and rock of definite types before final conclusions can be reached.

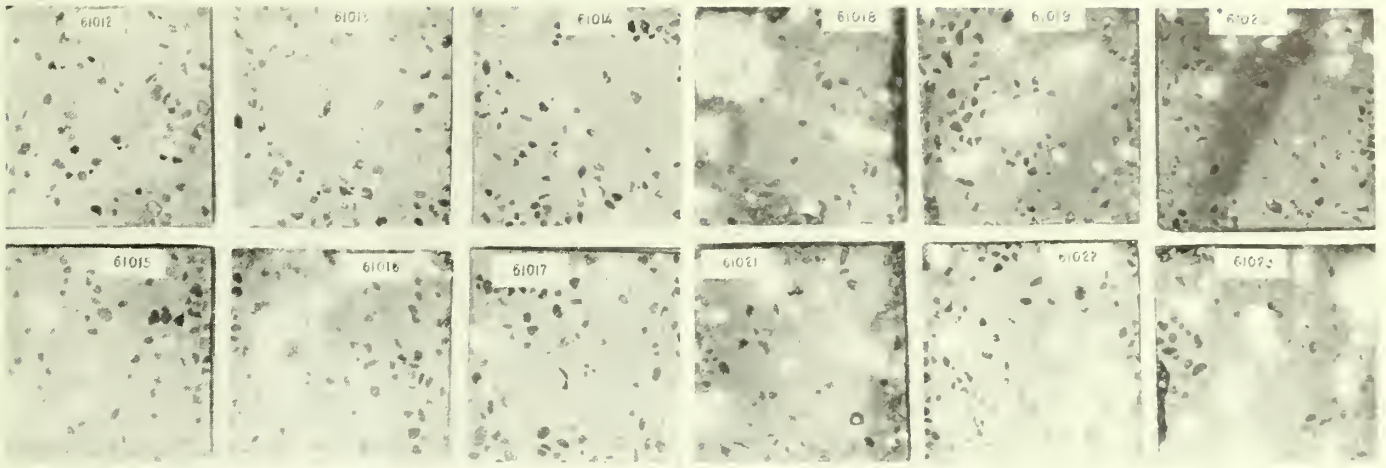

Figure 10.—Results of Pat Tests Using 12 Different Cements With Sand From Pocatello, Idaho, After 40 Days in Caustic Solution. Alkali Contents of Cements Ranged From 0.27 to 1.03 Percent.

Table 7.—*The extent of reaction on aggregates using cements of different alkali content after 40 days in caustic solution* [1]

Material	Source	Total alkalies in cements										
		1.03	0.97	0.96	0.82	0.78	0.74	0.72	0.67	0.60	0.52	0.35	0.27
Rock	Spray, Oreg	N	N	N	N	N	N	N	N	N	N	N	N
Sand	Pocatello, Idaho	M	M	M	S	M	M	M	S	S	S	M	M
Gravel	do	S	S	M	M	S	M	M	M	S	S	N	S
Sand	Cowlitz River in Washington	S	S	S	S	S	S	S	S	S	S	S	S
Gravel	do	S	VS	VS	N	S	S	N	S	N	N	N	N
Sand	White River in Washington	H	M	M	H	M	M	M	M	M	M	H	M
Gravel	do	M	H	H	H	M	M	M	H	S	M	H	H
Sand	Grandview, Wash	VS	VS	VS	VS	VS	VS	VS	VS	VS	VS	VS	VS
Gravel	do	N	N	N	N	N	N	N	N	N	N	N	N
Rock	Dixie School, Oreg	H	M	H	S	H	H	H	H	H	M	M	H
Sand	Burnt River in Oregon	S	M	S	M	N	S	M	M	S	M	VS	VS
Gravel	do	N	M	N	VS	VS	N	M	N	VS	N	N	N
Sand	Toutle River in Washington	VS	VS	N	VS	VS	N	VS	N	VS	VS	N	N
Gravel	do	N	N	VS	VS	VS	S	N	VS	N	N	N	N
Gravel	Spokane, Wash	N	N	N	N	N	N	N	N	N	N	N	N
Sand A	do	N	N	N	N	N	N	N	N	N	N	N	N
Sand B	do	N	N	N	N	N	N	N	N	N	N	N	N

[1] Reaction classified as: N=none; VS=very slight; S=slight; M=moderate; H=heavy.

MISCELLANEOUS TESTS

In the search for a clue as to the nature of the reaction product presumed to be formed by aggregate and alkalies, a pat of high alkali cement (1.15) was made in which were embedded pieces of wood opal. After proper curing, the pat was one-half immersed, in a horizontal position with aggregate particles on top of pat, in caustic solution and stored for about 8 months. At the end of this period, a heavy, sirupy, gelatinous material covered approximately two-thirds of the top of a 3- by 3-inch pat. Some of this gel was carefully removed and a chemical analysis made. The results are given below:

	Percent
SiO_2	66.5
Na_2O	19.8
K_2O	12.1
$Fe_2O+Al_2O_3$	0.9
CaO	0.4
MgO	0.3
Total	100.0

The analysis indicates that the gelatinous material is a silicate of sodium and potassium. It seems from all indications that the reaction gel may have been formed by one of three combinations, as follows: (1) Opal plus alkalies in cement; (2) opal, alkalies in cement, and caustic solution; and (3) opal plus caustic solution.

From the high silica content of the gel, there can be no doubt as to the source of this ingredient. Since the pat was only half immersed in the caustic solution, the sodium and potassium must have been derived largely from the alkalies in the cement, aided possibly by a small portion of the alkalies in the solution itself.

CONCLUSIONS

From the data obtained in this investigation and considering the test conditions, the following conclusions appear to be warranted:

The pat test is of little value as a short time laboratory test method for identifying reactive aggregates except in the case of opal-bearing materials. The test has value in detecting the presence of opal thereby indicating that further tests should be made to determine the degree of reactivity with any given cement. To be of diagnostic value for other than opal-bearing aggregates, the test period must be considerably more than 5 days.

Certain varieties of opal, such as wood opal and semiprecious opal are reactive with high alkali cements. Dehydrating the opal fails to delay its reaction with cement appreciably.

The pure silicon dioxide minerals, such as quartz, do not produce reaction when tested with high alkali cements.

Reaction has been found to take place in certain aggregates of types of the igneous, sedimentary, and metamorphic rock.

No reaction was produced in 220 days between wood opal and cements of low, medium, or high alkali contents when caustic solution in the concentration used in this test was used as "mixing water" instead of the regular tap water.

Analysis of the reaction product developed between wood opal and high alkali cement indicates that it is a silicate of sodium and potassium.

Certain aggregates react with low alkali cements as well as with cements of high alkali content.

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EVIDENCE IN WASHINGTON OF DETERIORATION OF CONCRETE THROUGH REACTIONS BETWEEN AGGREGATES AND HIGH-ALKALI CEMENTS. Journal American Concrete Institute, vol. 12, No. 6, June 1941, p. 673.

(*Continued from p. 46.*)

The following conclusions appear justified on the basis of the performance of the various test sections prior to construction of the bituminous wearing surface.

Surface defects such as corrugations, raveling, pitting, scaling, and potholing occurred on all sections regardless of thickness or physical properties of the mixtures. Resistance to abrasion on sections having plasticity indexes from 7 to 10 was no greater than on the sections having plasticity indexes of 4 to 6.

Surface wear resulting from the action of traffic amounted to about three-fourths inch per year.

A stabilized layer 1 inch thick cannot be satisfactorily maintained as a surface course when resting on the heavy clay typical of this road.

A 2-inch wearing surface on the heavy clay is likely to be worn out completely after 1 year of service owing to the fact that following the loss of 1 inch by traffic abrasion, the remaining 1 inch may soon be displaced in the manner described for section 1.

Stabilized sand suffers greater losses from abrasion than stabilized gravel.

The behavior of the reconstructed test sections when used as a base course seem to warrant the following conclusions:

A thin bituminous wearing surface such as the double blotter type seal requires a base course having a thickness of at least 8 inches to provide adequate support under the conditions of traffic, climate, and soils typical of this road.

With respect to the use of thicker bituminous surfaces, it was indicated that a re-treatment might have prevented the slight deformation and cracking starting to occur on section 7, which had a base thickness of 6 inches. The performance of the test sections constructed on the old bituminous mat, which was approximately 2 inches thick, suggests that a combined thickness of wearing course and base course amounting to 8 inches is required to give satisfactory service on the highly plastic clay soils of the Red River Valley. There was no definite indication that a lesser thickness would prove adequate on the relatively less plastic, lighter textured soils found on the eastern portion of the experiment.

It was clearly demonstrated that the service behavior of a stabilized sand base course was equal in every respect to that of a stabilized gravel base of the same thickness.

U. S. GOVERNMENT PRINTING OFFICE: 1944

ICE FORMATION ON THE ALASKA HIGHWAY[1]

Reported by WILLIAM L. EAGER and WILLIAM T. PRYOR, Highway Engineers, Public Roads Administration

THE ALASKA HIGHWAY extends from Dawson Creek, B. C., to Fairbanks, Alaska, a total length of 1,520 miles. Twelve hundred and twenty miles of the main highway is in Canada and 300 miles is in Alaska. Elevations range from 1,000 feet above sea level at the Muskwa River near Fort Nelson to 4,251 feet at the summit, 90 miles west of Fort Nelson. Most of the highway lies between elevations of 2,000 and 3,000 feet. A map and profile of the highway are shown on page 69. It extends from about latitude 56° to 64°, and from about longitude 120° to 146°. The area traversed is hilly or mountainous and for the most part is thickly timbered but the trees are generally too small to be of commercial value.

The highway was built during the period from March 1942 to November 1943. In 1942 seven regiments of U. S. Army Engineers, and 47 civilian contractors employing about 7,500 men working under the direction of the Public Roads Administration pushed through a pioneer road. Streams were bridged with temporary timber trestles not expected to withstand the spring break-up. The engineer troops were withdrawn from the highway before the beginning of the 1943 construction season with the exception of two companies that remained until July. Most of the permanent bridges required and an all-season gravel road suitable for heavy trucking were constructed during 1943 by 81 contractors employing about 14,000 civilian workers. These forces were directed by the Public Roads Administration.

REGION OF THE HIGHWAY HAS A SEVERE WINTER CLIMATE

The climate of the region traversed by the highway is classed as the subarctic and is characterized by relatively short and wet summers and long, cold, and dry winters. Spring and fall are short, inconspicuous seasons. Table I gives a comparison of the climate along the highway with other better-known places.

Figure 1 shows normal mean monthly temperatures along the route of the highway based on records of 5 years or more and compares these data with that for the past two winters. It will be noted that the maximum mean temperature variations occurred during the winter months. This condition seems to be typical for subarctic climates. Temperatures are apt to fluctuate widely during the same winter and from one winter to the next.

The few hours of sunlight in winter during which radiant heat is received and the long days in summer account for the wide spread in mean temperature. About midway of the highway between Watson Lake and Whitehorse, at latitude 60°, the sun is above the horizon for approximately 6 hours daily in midwinter and has an altitude of only about 7°. The highway receives practically no direct sunlight at that time. In midsummer the sun is visible for 19 hours and twilight extends through the night from April 23 to August 22.

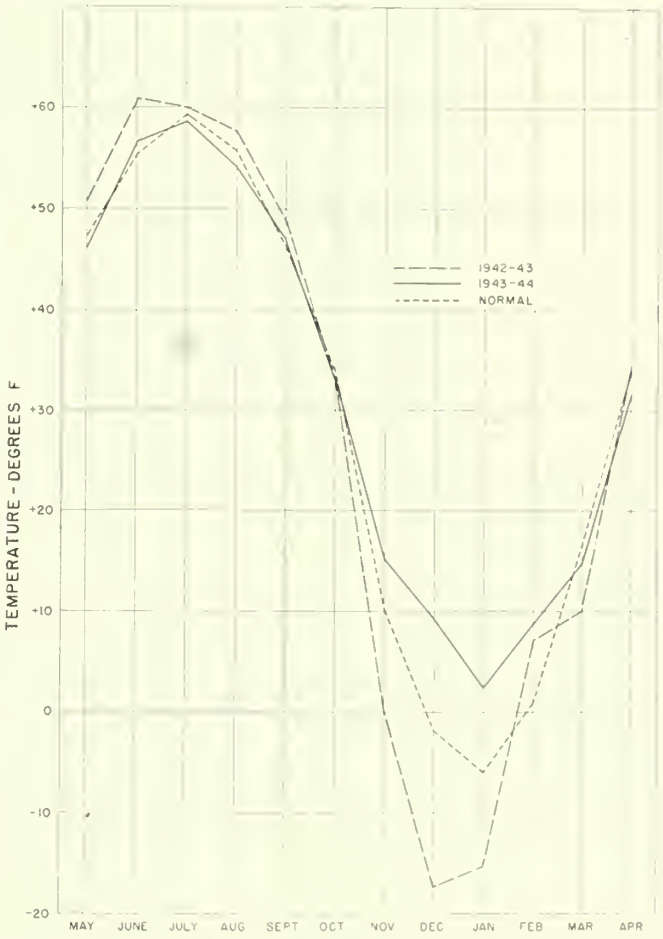

FIGURE 1.—MEAN MONTHLY TEMPERATURES ON THE ALASKA HIGHWAY BASED ON RECORDS AT FORT ST. JOHN, FORT NELSON, WATSON LAKE, WHITEHORSE, NORTHWAY, TANACROSS, BIG DELTA, AND FAIRBANKS.

[1] Prepared for the 1944 annual meeting of the Highway Research Board.

TABLE I.—*Climatological data* [1]

Place	Temperatures			Mean annual precipitation (water)
	Mean annual	Maximum	Minimum	
Along the Alaska Highway:	°F.	°F.	°F.	*Inches*
Fairbanks, Alaska	26	99	−66	11.8
Tanacross, Alaska			−76	
Whitehorse, Y. T.	27	84	−69	11.4
Fort Nelson, B. C.	30	98	−54	13.9
Fort St. John, B. C.	35	105	−65	15.7
Off the highway:				
Yellowstone Park, Wyo.	39	97	−66	20.4
Bismarck, N. Dak.	40	114	−45	16.3
New York City, N. Y.	52	102	−14	43.0
Denver, Colo.	50	105	−29	14.1
Los Angeles, Calif.	62	109	28	15.2
Miami, Fla.	74	96	27	55.7

[1] From the Statistical Abstract of the Bureau of the Census and from data obtained from offices of the U. S. Weather Bureau and the Canadian Department of Transport.

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About 60 percent of the precipitation in the areas traversed by the highway normally occurs from May through September. As these months are relatively cooler than they are at more southerly latitudes, the evaporation is less and the growth of vegetation is much more profuse than might be expected from the annual precipitation figures.

FORMATION OF ICE A SERIOUS OBSTACLE ON PIONEER ROAD

The type of ice formation described in this report is called icing and may be defined as the formation of ice in such a manner that its thickness and area are continually increased. It occurs when thin films of water flow out over the surface of the ground, snow, or previously formed ice during periods of subfreezing temperatures. These thin films of water quickly freeze and may result in a rapid build-up of ice. Unless prevented or controlled, ice formation may engulf the roadway and drainage structures and prevent or impede normal winter use of the highway.

In most areas of the United States, winter conditions are such that if ice does form, the water continues to flow under the ice and any additional freezing is from the bottom of this ice cover downward and little trouble from icing results. However, icing that builds up to considerable thickness is not entirely unknown along highways in the United States. Climatic, topographic, and drainage conditions along almost the entire route of the Alaska Highway are conducive to icing and it constitutes a major problem in winter maintenance. In this region the engineer must take into account the probable formation and effects of icing in locating, designing, and constructing highways if they are to be maintained and used during the winter and winter maintenance costs kept to a minimum.

At the time the pioneer road was constructed in 1942, it was anticipated that ice formation would cause some winter maintenance problems but the work required to keep traffic moving during the winter of 1942–43 was more difficult than was anticipated. The winter was colder than normal (see fig. 1). Wet, seepy areas drained by ditches had not drained out before the onset of winter. The grade of the pioneer road was generally low and adequate provision for drainage had not been made. Adequate equipment for the control of icing was not available.

Ice filled the ditches and culverts and formed on the road surface to considerable depth at many places. Crossings of the larger rivers of glacial origin such as the Donjek, White, and Robertson were practically impassable at times.

Some observations of ice conditions were undertaken the first winter both by the Army and by the Public Roads Administration. Major B. F. Hake of the Northwest Service Command reported on the icing conditions between Whitehorse and Big Delta. W. A. Keranen and R. J. Greisiger of the Public Roads Administration reported on conditions from Lower Post to the White River and in Alaska.

In constructing the final highway in the summer of 1943, 467 miles was relocated away from the pioneer road and 970 miles either coincided exactly or was placed approximately on the location of the pioneer road. In making relocations and in improving sections of the pioneer road to satisfactory standards, advantage was taken of the knowledge gained the previous winter about icing.

CAUSES OF ICING AND METHODS OF CONTROL STUDIED

The highway was relocated around a number of places where icing occurred; at places it was so constructed as to minimize the effect of ice formation, and at other locations water flow likely to cause icing was diverted away from the roadway.

After completion of the highway in October 1943, the U. S. Engineer Department assumed responsibility for maintenance. Most of the highway was maintained under contracts entered into with construction contractors. Advantage was taken of the experience in ice control gained the previous winter and the crews were much more adequately equipped and organized. This, combined with a milder winter and the greatly improved highway, resulted in keeping the roadway ice free and in excellent condition throughout the winter of 1943–44. The cover page shows a location where icing was kept under control by blasting to keep a ditch open.

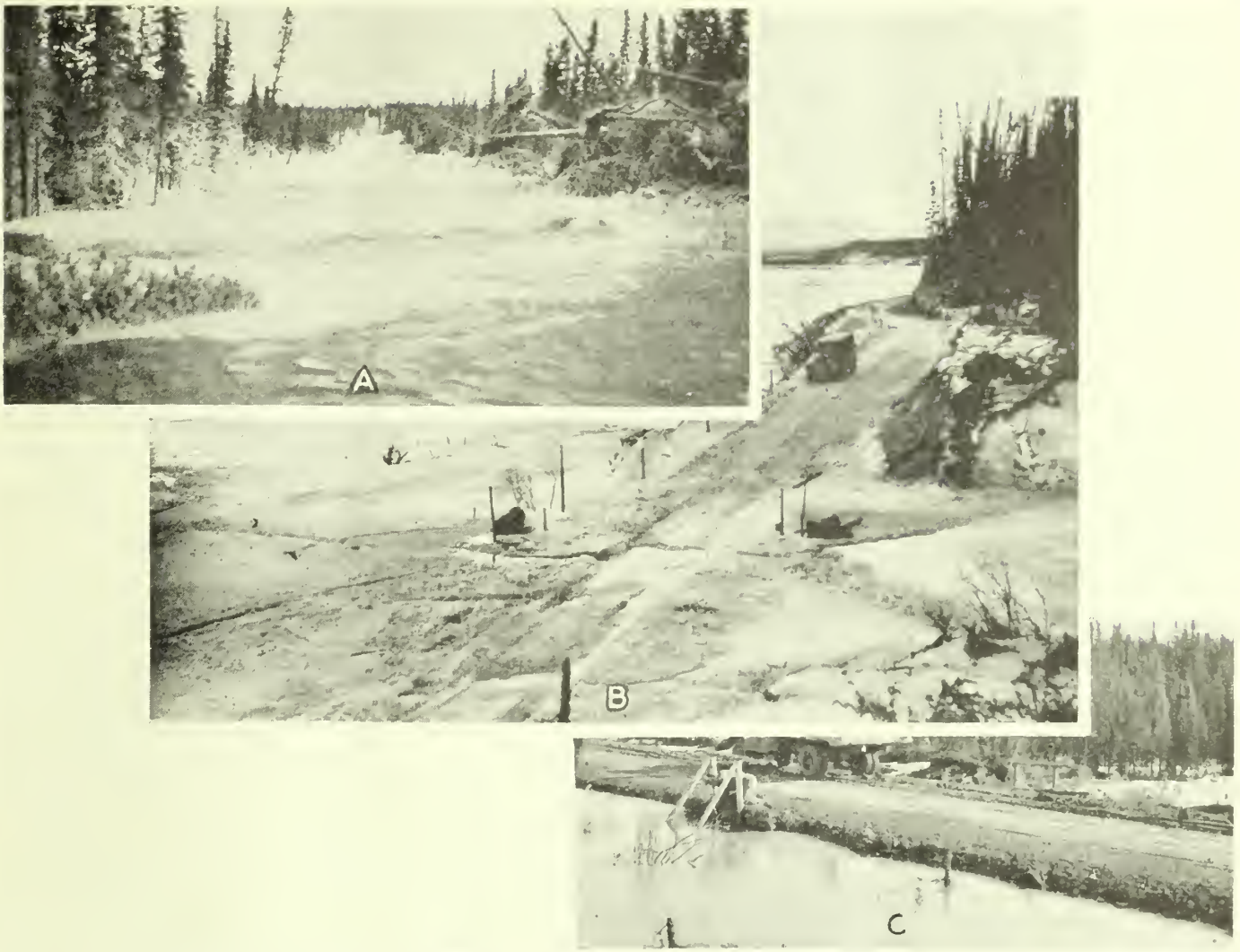
Since icing had been such a serious problem during the winter of 1942–43, it was desired to make a more thorough study of the matter during the winter of 1943–44. In accordance with a request from Thomas H. MacDonald, Commissioner of the Public Roads Administration, the Division Engineer of the U. S. Engineer Department, Brigadier General L. D. Worsham, granted permission for the Public Roads Administration to carry on icing studies during the winter of 1943–44 as a research project. The information obtained in this study was made available to the maintenance forces on the highway, and the engineers assigned to this work served as consultants with the objective of improving the icing prevention and control methods used.

Preliminary icing investigations and studies were made during the summer and fall of 1943 under the direction of S. E. Horner, geologist of the Public Roads Administration. Studies during the 1943–44 winter were carried on by the writers of this report with the assistance of R. J. Greisiger, highway engineer, whose experience along the Alaska Highway the previous winter proved of great value.

The studies were carried out with the intention of obtaining information for use in maintaining the Alaska Highway and in construction and maintaining other roads where similar conditions exist.

PHENOMENA OF ICING DISCUSSED

When thin films of water are exposed to very low temperatures such as are common along the Alaska Highway in the winter, a very rapid build-up of ice may occur. Generally speaking, icing cannot be prevented entirely by any method that is reasonable in cost but under certain conditions it may be controlled so as not to interfere with the use of the highway. It is important to know what causes the water to emerge on the surface in thin films, since once it emerges icing will result if the temperatures is low enough. In the area of the Alaska Highway the temperatures are generally below freezing throughout the winter and the surface insulation in the form of snowfall is generally light. Ice forms on the surface of water in normal drainage channels and the water continues to freeze downward from the surface, inward from the sides of the channel, and even occasionally upward from below. Ice may so constrict a channel that the water is forced by hydrostatic pressure to break out on the surface.



ICING ON THE PIONEER ROAD IN WINTER OF 1942–43: A, ON BRANCH ROAD TO HAINES NEAR ITS JUNCTION WITH THE ALASKA HIGHWAY; B, ALONG THE KLUANE RIVER, SEVERAL FEET OF ICE HAS FORMED ON THE ROAD. NOTE THE BARRELS FOR BURNING OIL; C, THAWING A CULVERT IN EARLY SPRING.

Icing can occur only when the latent heat contained in water is transferred to some colder medium. There are three methods of heat transfer—radiation, convection, and conduction—and all of them are important in icing.

Radiation.—Solar radiation—past or present—is the source of practically all our heat and energy. Terrestrial radiation from the earth outwards into space maintains the heat balance so that the earth as a whole remains at a livable temperature. During winter in the higher latitudes, the receipt of solar radiation is greatly decreased while the loss of heat from the earth by terrestrial radiation goes on practically unabated. This process cools the surface of the earth and therefore the air which is in contact with it. Precipitation occurs in the form of snow rather than rain. A snow covering on the ground further cools the air since snow is both a good reflector and a good radiator. It does, however, serve to conserve the heat in the underlying earth or water as it is a good insulator. Thus the air temperature in winter in the higher latitudes is apt to be very low and any flow of water exposed to air may quickly freeze. The rate of change in air temperature throughout the year follows closely that of receipt of solar energy. Figure 2 shows the variation in radiation received and the variation in mean monthly temperatures at Fairbanks, Alaska.

Convection.—Convection currents in water or in the air account for a large part of the transfer of heat involved in freezing water to ice. The warmer air or water tends to rise since it is less dense while the colder particles settle. However, in the case of air an additional complication enters since air expands as it rises and this expansion tends to cool the air. Thus the air over most of the world is in continual motion, particularly during the warmer seasons of the year. In arctic or subarctic regions the cooling of the air next to the earth's surface (as described under radiation) tends to establish an equilibrium and the lowest temperatures are found in the valleys or on the plains.

In pure water the maximum density occurs at a temperature of 39.2° F. so that as the surface of a body of water becomes cooler, by losing heat by radiation and conduction, the cooler water becomes denser and therefore sinks and is replaced at the surface by warmer, less dense water. This process continues until the entire body of water reaches a temperature of 39.2° F. As cooling progresses beyond this point the colder water is less dense and remains on top, eventually reaching a temperature of 32°. It remains at this temperature

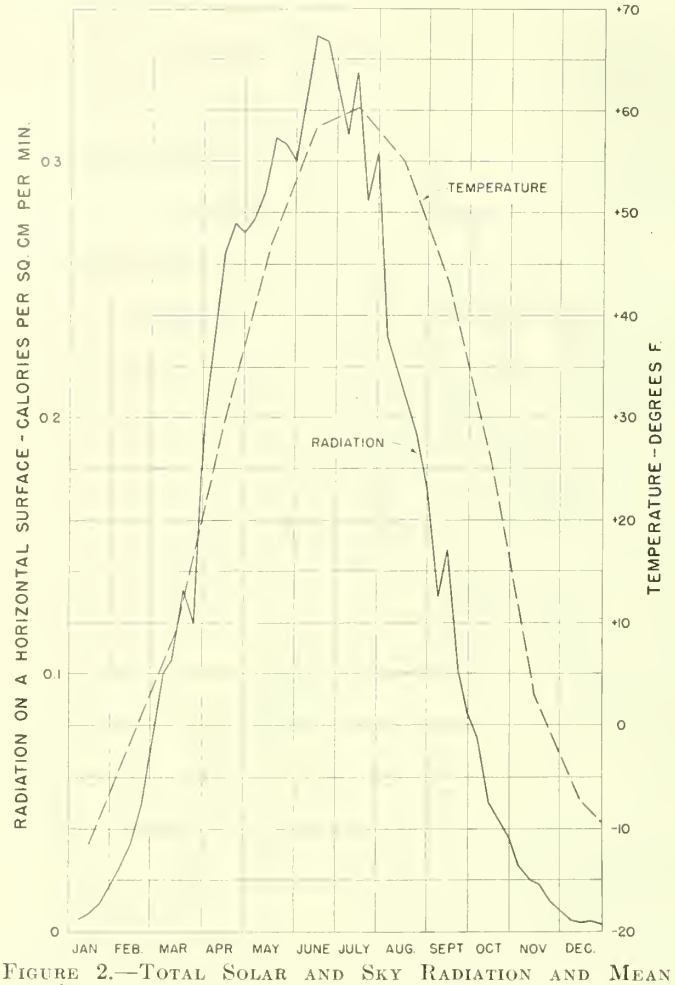


FIGURE 2.—TOTAL SOLAR AND SKY RADIATION AND MEAN TEMPERATURE, BY MONTHS, AT FAIRBANKS, ALASKA.

until it has lost its latent heat and turns into ice which thereafter grows by freezing downward. The ice covering may then have a temperature differential varying between 32° on its lower surface and whatever the air temperature is on its top surface.

In swift, turbulent streams the normal action described above is prevented. Rapid mixing of the water takes place with the result that the entire flow will at all times be at approximately the same temperature. The temperature remains at approximately 32° F., until the water all freezes to ice or until a surface cover of ice forms. In swift streams this action is conducive to the formation of anchor and frazil ice.

Conduction.—Conduction is the transfer of heat by vibration of the molecules of the substance through which the heat is being transferred. In the action of icing conduction is an important means of heat transfer through all solid substances and the rates at which it is transferred are more readily determined than the rate by which it is transferred by convection through liquids or gases.

Water in contact with colder solid substances loses heat largely by conduction to these substances. The heat contained in water may be transferred by conduction and convection through and from the water to the substances with which it is in contact such as ice, snow, air, ground, and vegetation. Once the heat contained in the water (at least 80 calories of heat per gram must be removed before water turns to ice) reaches the surface it may be lost directly by radiation into space or it may warm air and be distributed elsewhere by convection currents. The rate of heat transfer by conduction increases with increase in the temperature differential between the air and the water with which it is in contact. Water cannot ordinarily exist as such at a temperature lower than 32° F., but the substances in contact with the water may be at temperatures as low as that of the air. On the Alaska Highway temperatures as low as −76° F. have been recorded.

Ordinarily the coldest medium with which water will come in contact is the air and that air is cold principally because it has been in contact with the surface of the earth or substances or objects on the surface of the earth which have lost heat by radiation into space. Air itself has no great capacity for storing heat and it is a poor conductor but conduction to very cold air and convection currents will remove large amounts of heat from a water surface. Since a water surface radiates heat nearly like a black body, much heat may be lost directly by radiation into space if the sky is clear and the air has a low moisture content. Radiation back from the sky is largely controlled by the moisture content of the atmosphere.

Ultimately the heat lost by water when it freezes is lost to the air by conduction or radiation or is radiated into space. This heat loss may be directly from the water or it may be through intermediate substances in contact with the water. These substances generally act to retard the loss of heat from the water. The thicker the covering of ice on a lake or stream the greater the insulating effect and the slower the rate of freezing. Snow has an insulating effect 4 to 20 times that of ice depending upon its degree of compaction. The ice over a body of water provides a support for a snow layer so that under normal conditions the depth of ice covering tends to become stabilized. This is fortunate else many streams and lakes would freeze solidly to the bottom resulting in the destruction of much animal life.

SNOW ONE OF THE BEST INSULATORS

Reference to typical conductivity constants indicates that heat is transferred by conduction through water about 20 times faster than through air, and through ice about 4 times faster than through water. The rate of conduction through ice is slightly faster than through earthy materials such as moist soil and the earth's crust and from 4 to 20 times faster than through snow, depending upon the age and compaction of the snow. The rate of conduction through water or air is not highly important, since convection in these materials will affect a much greater transfer of heat than will conduction alone.

It is apparent that if icing is to be prevented or retarded, the loss of heat—by any method—must be prevented or retarded or the lost heat replaced from some other source. Of the materials readily available in an icing area, snow is one of the best insulators. This explains why icing is not a serious problem in areas of heavy snowfall, even with very low temperatures. While ice is not a particularly good insulator it does prevent direct radiation of heat from underlying water and serves as a support for subsequent snow cover—either naturally or artificially placed. The vegetation which grows so profusely in the region of the Alaska Highway has excellent insulating properties and should not be disturbed any more than necessary if

icing in a watercourse is to be minimized. At some places, it may be desired to cause icing, so that water will freeze before it reaches the vicinity of the highway. At such places conditions which would naturally insulate the water must be altered.

OCCURRENCE AND DEVELOPMENT OF ICING DESCRIBED

As the winter progresses there is freezing at the sources of flowing water. Frequently this results in a decrease of channel flow after the surface has been frozen over. Air spaces will then occur between the surface of the water and the ice and the loss of heat is apt to be materially decreased since confined air is an excellent insulator. This action occurs in both large and small channels.

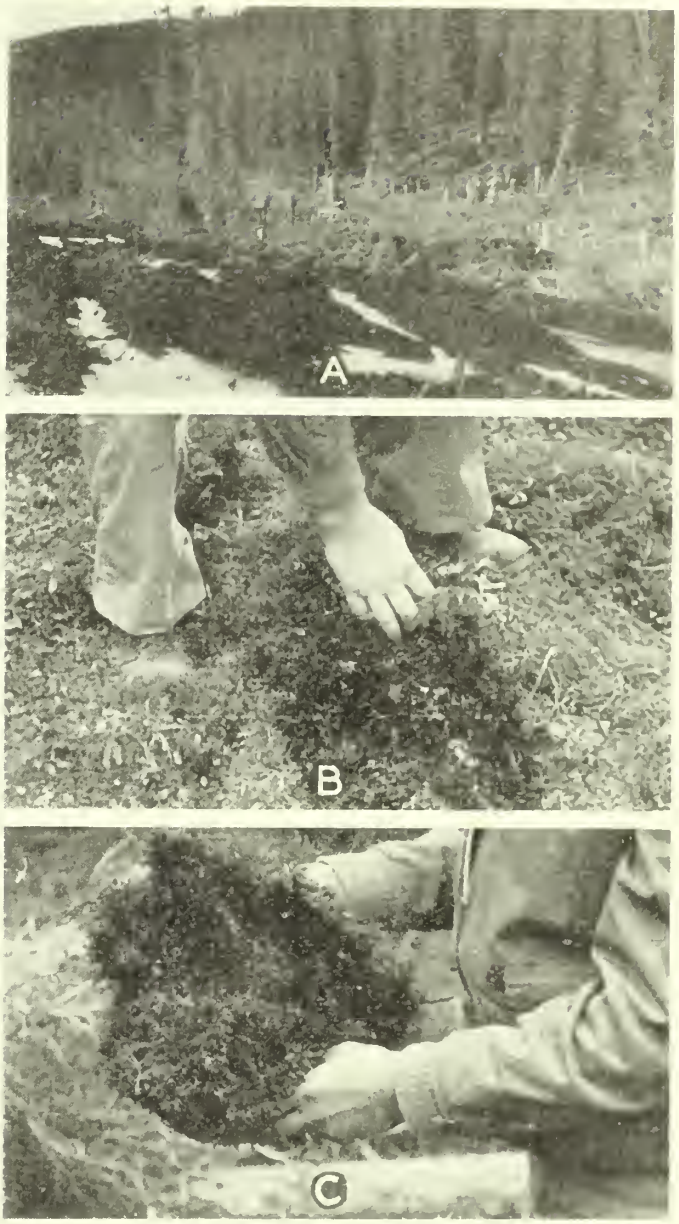
Moss, vegetal debris, and down timber over an area will act as insulators. It is difficult to say what the relative insulating values of these materials are, but undoubtedly the materials are effective, particularly when supporting a fluffy covering of snow. Moss, low bushes, and grasses, when densely matted, make such a good insulating combination that a thickness of 12 to 18 inches will preserve underlying permafrost during the summer and will prevent water from freezing near the roots of the vegetation during the winter. Just how this water is prevented from freezing with frozen material both above and below is something not explained, yet this condition was frequently observed along the Alaska Highway. Water that continues to seep or flow from such areas is responsible for a large part of the icing along the highway. It may be possible that organic decay in the mass of dead roots and vegetation generates enough heat to keep the water from freezing.

The water flow which eventually emerges on the surface and results in icing on or near the highway, may come from any one or all of three sources: (1) Surface water flowing in rivers, creeks, and small streams, the source or sources of each being at a considerable distance from the icing; (2) spring water from fissures and porous strata flowing to the surface at a definite place on or near the highway, and; (3) percolating water or seepage from muskeg swamps, talus slopes, alluvial fans, seams between strata of ledge rock, and sloping ground with a heavy vegetal cover.

The term "seepage" is used here in a limited sense to describe water surfacing on or near the highway that does not have a single discernible surfacing place. The term "muskeg swamp," as used in the region of the Alaska Highway, refers to any basin which does not readily drain and which is filled with saturated muck, fine silty soil, decayed vegetation, and has a heavy ground cover of moss, grasses, and low-growing bushes. There are frequently scattered growths of stunted trees on these areas. These swamps frequently occur on hillsides between ridges, where, in spite of the slope, the muck and vegetation retain a high concentration of water. Frequently, the subsoil in these swamps is permanently frozen.

ICING PROBLEMS AT RIVER CROSSINGS SOLVED

During the winter of 1942–43 the most serious icing conditions, from the standpoint of travel, developed at some of the rivers and larger streams which were crossed at low level on temporary timber trestles. It was evident at this time that the permanent bridges should be placed above any possible ice formation, and

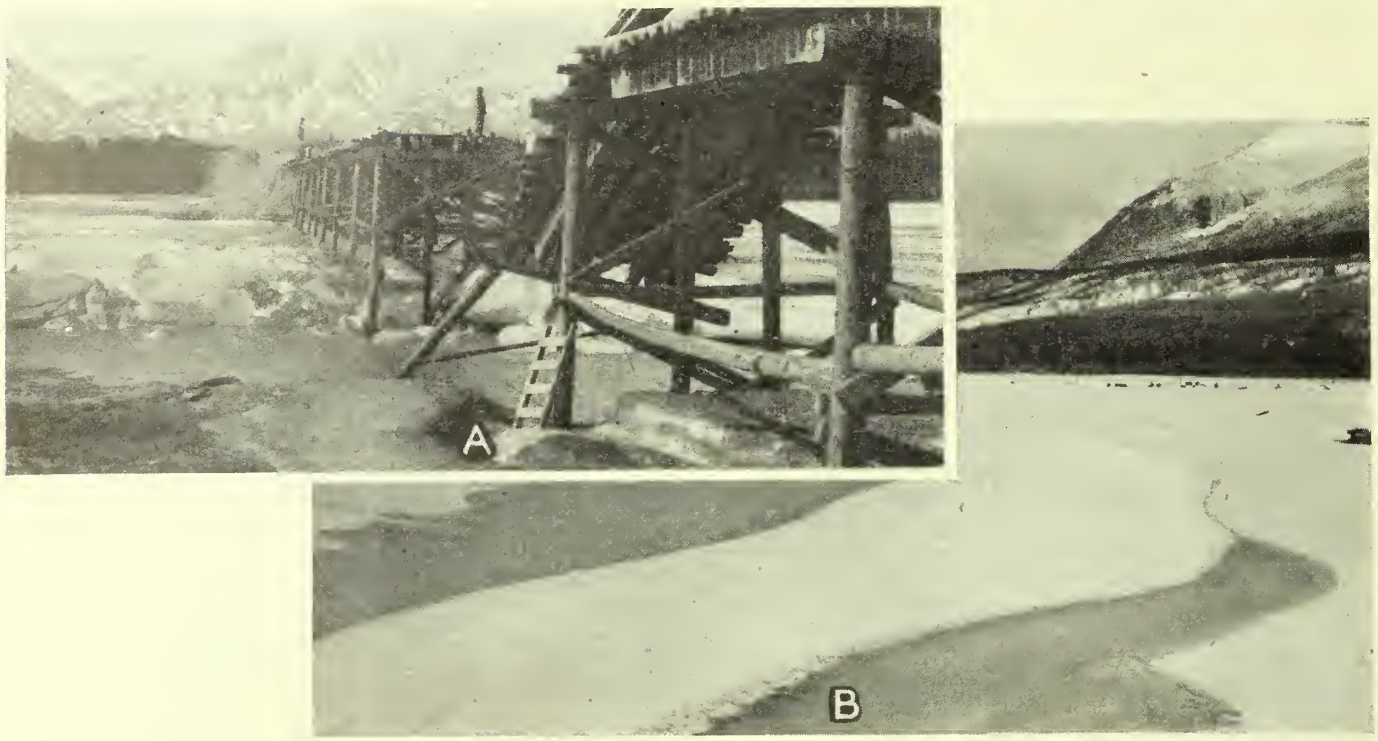


A, Natural Conditions Conducive to Icing. The Picture Was Taken in October 1942 as Winter Began. B and C, Ground Cover of Moss and Vegetal Debris That Prevents Evaporation in Summer and Freezing in Winter.

the designs for the permanent structures were prepared accordingly.

During the 1943–44 winter, after the permanent crossings had been constructed, no trouble was experienced at any of these locations. Ice in such rivers as the Donjek and the Robertson developed to about the same level as the winter before. At the Donjek, one of the few large streams where timber trestles remained in use, ice formed to within 1 foot of the bottom of the stringers. The original crossing here had been replaced by a series of timber trestles, at somewhat higher level, separated by gravel fills. At the Robertson, which was crossed all winter on a temporary trestle, ice did not form high enough to approach the road level. Floating ice in the spring knocked out several bents.

Icing on the larger streams has been successfully coped with by placing the superstructures above the level of



A, Floating Ice Took Out Two Bents of Temporary Trestle Across the Robertson River. Ice Resting on Gravel Beds Has Formed to a Considerable Height Above the Summer Stream Level. B, Ice Blister in the Donjek River Caused by Hydrostatic Pressure in the Constricted Channel. Water Has Emerged and Will Soon Freeze.

all possible ice formation, and by making the substructures substantial enough to withstand the effects of floating ice during the spring break-up. In streams like the Robertson and Johnson Rivers, ice builds up in successive layers as a result of overflow and freezing on top of previously formed ice. The ice layer is so thick that it rests on the stream bed rather than floats on the water. Consequently most of it melts in place the next spring and summer and is not carried downstream in blocks. Natives say that the ice "rots out." In other streams with broad surfaces, like the Peace, Sikanni Chief, Muskwa, and Liard Rivers, thick ice forms by downward freezing but does not build up greatly on the surface first frozen. This ice goes out in a wild charge during the spring break-up.

The remainder of this article is devoted almost entirely to icings caused by seepage water or flowing streams not more than a few feet wide, although there is occasional reference to the action in larger streams.

ICINGS CLASSIFIED

Icings along the Alaska Highway may be divided into two classes.

Natural Icings.—These would develop normally regardless of road construction. They occur as follows:

(1) In glacial rivers and creeks and in streams on alluvial fans, where the nature of the channel is such that little protection from freezing is afforded the flow. Relatively steep gradients are a factor, since channels under the ice become constricted by continued freezing, and water is subjected to considerable hydrostatic head, often breaking through the ice cover and spreading out on the surface. Icing is always worse in a wide, shallow, gravelly channel with a steep gradient than in a deep, narrow, low-velocity stream having a heavy overhanging growth of vegetation along the banks.

(2) Where seepage and subsurface water flows from springs that reach the surface near the base of a hill or mountainside. Wherever water emerges in small quantity it is sure to freeze. The additional exposure at the roadway with the resulting greater depth of frost penetration may be a factor in forcing the flow to the surface. However, it has been noted that ice frequently forms where water emerges at locations remote from any influence of the highway construction.

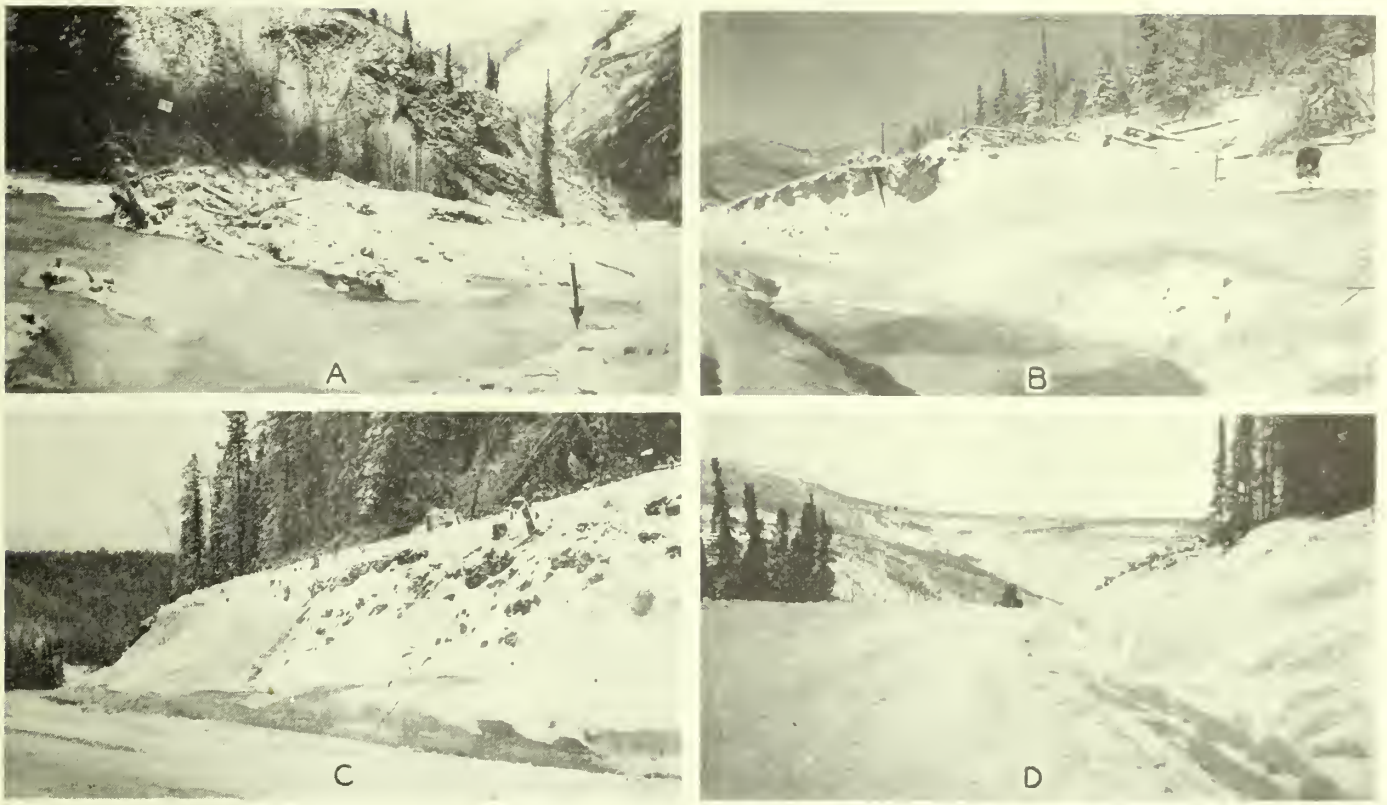
(3) At light surface flows down steep watercourses. Here the flow is spread out thinly to give maximum exposure to the atmosphere and loss of heat by radiation.

Artificial Icings.—These are caused or promoted by the road construction. They occur as follows:

(1) At seeps and springs intercepted by highway excavation. Springs are more apt to be found at considerable depths and therefore are likely to continue active all winter. Seepage flows most often appear in the lower part of or immediately under a heavy vegetal ground cover. With prolonged periods of cold weather and only a light snow cover, these seepage flows may freeze up before they reach the roadway and become inactive.

(2) At crossings of small streams where the additional exposure created by the construction promotes the formation of ice. These are narrow, deep, low-velocity streams with heavy vegetation on banks which normally do not ice—at least not at the point crossed by the highway.

Icing may be started by freezing in a culvert or under bridges or at a disruption or obstruction of the normal drainage channel. Since the culverts and bridges are invariably placed under or in fill materials of high heat conductivity, and snow covering on the road surface is removed by maintenance forces, the opportunity for heat loss is greater than in the original channel. The



A, Ice Formation Where a Small Stream Enters a Culvert. The Arrow Indicates the Entrance to the Culvert. B, Typical Icing in a Waterway and Roadside Ditch. C, Ice Formed in the Ditch and on the Back Slope. D, Ice Had Begun to Encroach on the Roadway When It Was Loosened and Bladed Across the Road.

normal, protected channels are necessarily disturbed within the limits of the construction operations and may even be unnecessarily obliterated or obstructed in these operations.

(3) At highway fills where subsurface flows are forced to the surface by a damming action. This may result from additional compaction but more often removal of protective cover in construction and maintenance of the highway permits frost to penetrate much more readily and thus create a "frost dam." Considerable icing occurred where very light fills were placed.

The mass which forms in icing action has the appearance of a heavy viscous liquid in motion. The formation of ice in relation to the supporting earth is much the same as that of a lava flow, although the methods of formation are entirely dissimilar. Small obstructions or restrictions in the channel cause the ice to dam up in its downstream progress.

CONDITIONS UNDER WHICH MAJOR ICINGS OCCUR ANALYZED

In the study of icing conditions along the Alaska Highway during the winter of 1943–44 the highway was divided into two sections, since one man could not travel and inspect icings on the entire highway with sufficient frequency, even though the highway could be traveled without difficulty throughout the winter. One section extended 913 miles from Dawson Creek to Whitehorse. The other section included the 509 miles to Big Delta, approximately 100 miles from Fairbanks.

At 92 places between Dawson Creek and Whitehorse the icing was active enough to constitute a considerable maintenance problem and was classified as major. There were 34 such places between Whitehorse and Big Delta. In addition, there were 57 places between Dawson Creek and Whitehorse and 38 between Whitehorse and Big Delta where minor icing activity occurred. Observations of all these icing points were made on various inspection trips during the winter. Information on icing activity and maintenance methods used were obtained by discussion with the maintenance men and from written reports prepared by them and by officials. In the spring after the snow and ice had largely disappeared a close inspection was made of the drainage conditions and other features at each of the major icing points.

Practically none of the icings observed could be attributed wholly to a single circumstance or condition. More than half occurred where natural conditions would have produced some icing had they been left undisturbed but the amount of icing was greatly increased as a result of disturbance in construction of the highway.

There follows a summation of the data concerning conditions at places where major icings occurred. Following this presentation the effect of the conditions will be discussed.

Natural and Artificial Conditions.—At 65 percent of the icings the natural slope of the drainage channel was conducive to icing and 49 percent of the icings formed where subsurface flow emerged naturally near the foot of a hill or mountain. Increased exposure of water to freezing by removal of ground cover or alteration of channels in construction of the highway contributed to over 90 percent of the icings.

The original, natural drainage channels at the roadway were classified as follows: 13 percent good, 37 percent fair, 31 percent poor, and 19 percent were not classified. Of the constructed drainage channels at the

roadway, 2 percent were classed as good, 27 percent fair, 46 percent poor, and 25 percent were not classified.

Source of Water.—Flow of water to points of icing sometimes came from more than one source. Percentages of icings by water sources were as follows:

	Percent
Creek	6
Small stream or branch	54
Spring	14
Seepage	44

Road Location.—Terms descriptive of road location are applicable at points of icing as follows. At some locations two terms are applicable.

	Percent
Across flats and at a considerable distance from a hill or mountainside	21
Rolling, irregular grade	34
Sidehill cut and fill	30
At or near foot of hill or mountainside	33
Miscellaneous	27

The cross slopes at the major icing locations are grouped as follows:

	Percent
Nearly flat	20
Light, up to 15 degrees	45
Heavy, over 15 degrees	35

Subgrade Soil.—Considering the length of the entire highway, 43 percent was located on soils classed as better than A–3–4. These better subgrades were mainly A–2, A–2–3, A–2–4, A–2–7, A–2–3–4, rock and gravel. The subgrades worse than A–3–4, found on 57 percent of the highway, were classified as A–4–7, A–5–7, A–7, A–4–6, A–2–4–6, A–4, A–6–7, A–8, and A–4–6–7.

The distribution of points of icing as to character of soil was as follows. A few places had two types of soil and were placed in two classifications.

	Percent
Sand, sand and gravel, rock	45
Silt with some sand or gravel	23
Silt with little or no sand or gravel	9
Clay with some sand or gravel	11
Clay with little or no sand or gravel	9
Silt and organic soil, A–5, A–8	11

Type of Culvert.—On the entire highway 66 percent of the culverts were made of wood and 34 percent were metal. The points of icing were distributed as follows:

	Percent
Wood culvert	46
Metal culvert	40
Both wood and metal culverts	13
No culvert	1

Ground Cover.—Practically the entire region of the Alaska Highway below the timber line is covered with a thick mat of moss and vegetal debris. With regard to thickness of ground cover the points of icing were distributed as follows:

	Percent
Heavy, over 12 inches	72
Medium, 6 to 12 inches	25
Light, 6 inches or less	2
None	1

Months of Icing Activity.—During the winter of 1943–44 major icings were active as follows:

	Percent
November	31
December	62
January	76
February	75
March	54

SEEPAGE AND SMALL STREAMS CAUSE MOST ICINGS

Icing occurs in a natural channel when it is wide and shallow, has little protection by vegetal growth, and a fairly steep gradient or a sudden change in gradient. Near the foots of hills and mountains subsurface flows tend to be forced to the surface and icings are likely to occur at such places.

At practically every icing the greater exposure resulting from construction of the highway has been a factor. Greater exposure cannot be avoided but can be compensated for to a certain extent by improvement of the natural drainage channels. Methods will be discussed later in this report. In making most roadway cuts it is impossible to avoid altering the natural drainage channels and intercepting new ones. However, many of the natural drainage channels were obstructed by debris pushed aside in clearing the roadway or with excavated material. Some drainage structures were so located as to disturb the continuity of the natural drainage course.

On the whole it must be expected that numerous locations naturally subject to icing or which will be made subject to icing by construction of the road will be found. If winter maintenance is to be kept within reasonable bounds some compensation must be made in the design of the roadway and drainage channels.

Streams from about 1 to 4 feet wide and from 3 to 12 inches deep and seeps are the principal sources of water causing major icing. In most cases the small streams originate in springs or seep from muskeg areas or from under a heavy mat of ground cover. Seepage most often emerged near the tops of cut slopes. A heavy protective layer of moss and vegetal debris and sometimes snow prevented the water from freezing before reaching the highway.

The seepage, in addition to supplying water that froze on emerging on or near the highway, produced conditions that afforded a considerable degree of protection of the water until it reached the highway. Trees and other vegetation grow more densely in the presence of an ample water supply and these produced a thick ground cover. This cover not only protected the water from freezing in winter but in summer it prevented lowering of the level of permafrost.

The data on road location and on cross slope at icings indicate that there is less of a tendency toward icing where the highway is located on relatively flat ground. Conversely, a fairly steep watercourse gradient is conducive to icing.

Observations on the direction of exposure of icings indicated a definite tendency for icings to be more active on southerly slopes. Apparently the additional heat from the sun keeps some small water flows active yet this heat is not sufficient to prevent icing where the flow is exposed to cold air at the roadway. The direction of exposure is not important at midwinter since practically none of the highway is exposed to direct rays of the sun at that time.

The data on type of subgrade soil indicate a tendency for ice to form where the subgrade material is of the more porous or granular type. Obviously such soil is better able to carry subsurface water and is most commonly deposited along stream courses.

It has been suggested that wooden culverts, because of their better insulating properties, might have less tendency than metal to cause freezing in the winter. The observations made do not indicate that either material is superior to the other in this respect.

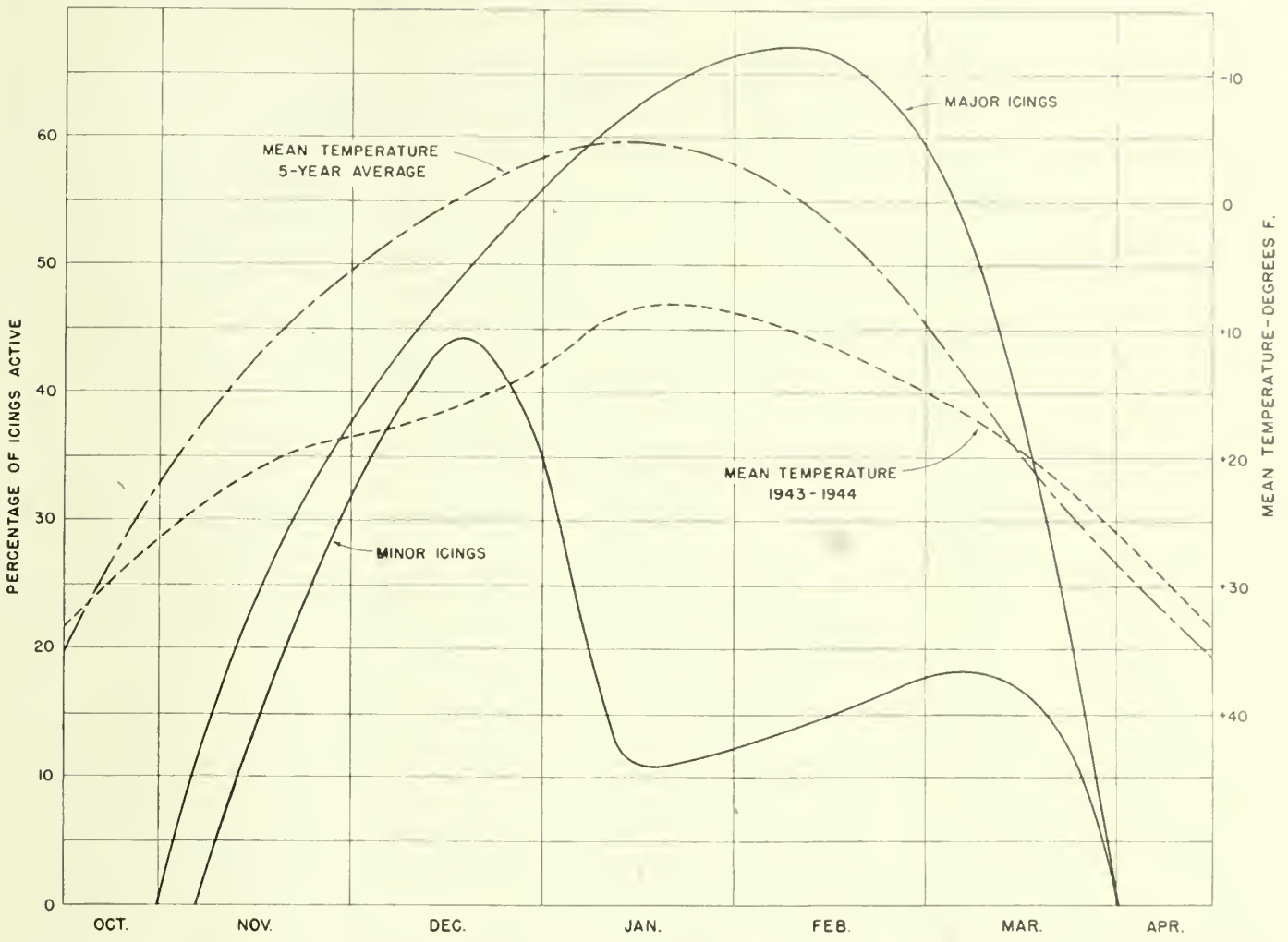


FIGURE 3.—PERCENTAGE OF ICINGS BETWEEN PROCHNIAK CREEK AND WHITEHORSE ACTIVE, BY MONTHS. ALL DATA ARE FOR THE WINTER OF 1943–44 WITH THE EXCEPTION INDICATED. THE CURVE MEAN TEMPERATURE, 5-YEAR AVERAGE, IS BASED ON RECORDS AT WHITEHORSE, WATSON LAKE, FORT NELSON, AND FORT ST. JOHN FOR A PERIOD OF 5 YEARS OR MORE. TEMPERATURE DATA FOR 1943–44 ARE BASED ON RECORDS AT SIX STATIONS BETWEEN PROCHNIAK CREEK AND WHITEHORSE.

PERMAFROST A FACTOR IN PROMOTING ICING

Permafrost is permanently frozen ground, or ground in which the temperature remains below freezing throughout the year. If the ground contains enough moisture, the soil particles will be bound together in a hard, impermeable mass. If little moisture is present, the materials may be loose even though the temperature remains below freezing. Along the Alaska Highway all permafrost observed had a high ice content. The upper level of frozen material commonly varies somewhat with seasonal changes in temperature and may, in the winter, meet the lower level of surface freezing. Very little permafrost occurs south of the upper crossing of the Liard River at mile 642, apparently because the average temperatures are higher at the lower latitudes. From observations along the route of the highway it appears that the mean annual temperature must generally not exceed 28° F. if permafrost is to be maintained.

Table 2 shows the occurrence of permafrost and of icing activity on the 437-mile section of highway south of Whitehorse to Prochniak Creek. The greatest amount of permafrost was encountered between Nisutlin Bay and Teslin River. This table indicates a decided tendency for icing to occur on areas of permafrost.

TABLE 2.—*Occurrence of permafrost and of icing on 437-mile section of highway south of Whitehorse*

Highway section	Length	Over permafrost					
		Highway		Major icing		Minor icing	
	Miles	*Miles*	*Percent*	*Number*	*Percent*	*Number*	*Percent*
Prochniak Creek to Lower Crossing	16.4	0	0	0	0	0	0
Lower Crossing to Upper Crossing	146.2	.5	3	0	0	1	7
Upper Crossing to Nisutlin Bay	160.9	7.0	4.4	1	1	3	6
Nisutlin Bay to Teslin River	33.1	9.0	27.2	9	64	3	30
Teslin River to Whitehorse	80.3	2.0	2.5	0	0	0	0
Total	436.9	18.5	4.2	10	17	7	8.5

From Whitehorse to Big Delta 28 percent of the road was constructed over ground classed as permanently frozen at the time of construction. Sixty-eight percent of all the major icings in this section occurred on these areas. Permafrost appears to be a factor in promoting icing activity. It is evident that permafrost layers hold the water near the surface where it can readily freeze instead of allowing it to soak away to protected depths.

ICE DAM FORMED OVER PERMAFROST CAUSES TROUBLE

An interesting icing condition occurred on a 3-mile section of the road west of Tanacross in Alaska. Here,

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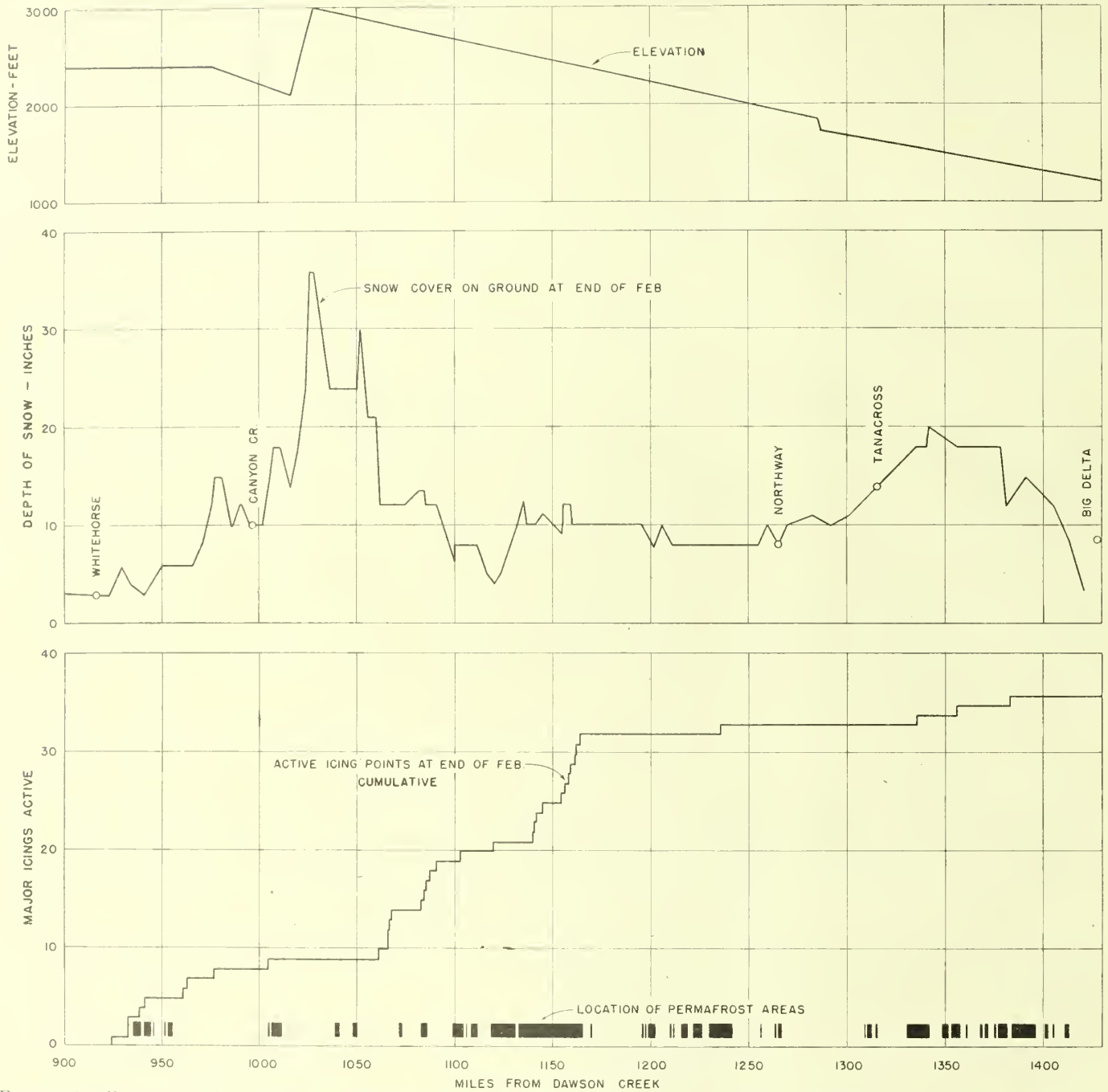


FIGURE 4.—ELEVATION, SNOW COVER, ACTIVE ICING POINTS, AND PERMAFROST BETWEEN WHITEHORSE AND BIG DELTA. ALL DATA ARE FOR 1943–44.

the roadway crosses several miles of a rather flat, swampy ground. There is a slight slope to the ground in a direction at right angle to the highway and some water normally seeps down slope through the grass roots and in the porous layer of decayed vegetation immediately underneath. The swamp is crossed on a gravel fill about 3 feet deep and there are numerous cross culverts. The vegetation is underlain everywhere at a depth of a foot or so with permanently frozen ground.

During the winter of 1943–44 a solidly frozen dam of material developed at the highway, extending from the roadway surface down to the permafrost layer below. This sealed the normal seepage channels and left the cross culverts as the only passageway for the flow. Since these culverts were in a relatively exposed location they quickly froze, and flooding developed over the swamp area upward from the highway. This flooded area then froze over, the ice forming around the vegetation. Continued flow of water under the impermeable cover of ice developed hydrostatic pressure which lifted the entire mass—grass, bushes, small trees, roots, and ice cover—as much as several feet. This action did not take place uniformly over the entire area but produced a series of small hummocks. Trees growing on these hummocks were shifted from a vertical position, resulting in what is called by the Russians a "drunken forest."

Water flow from ruptured places in the ice cover caused additional flooding and icing and, for a time,

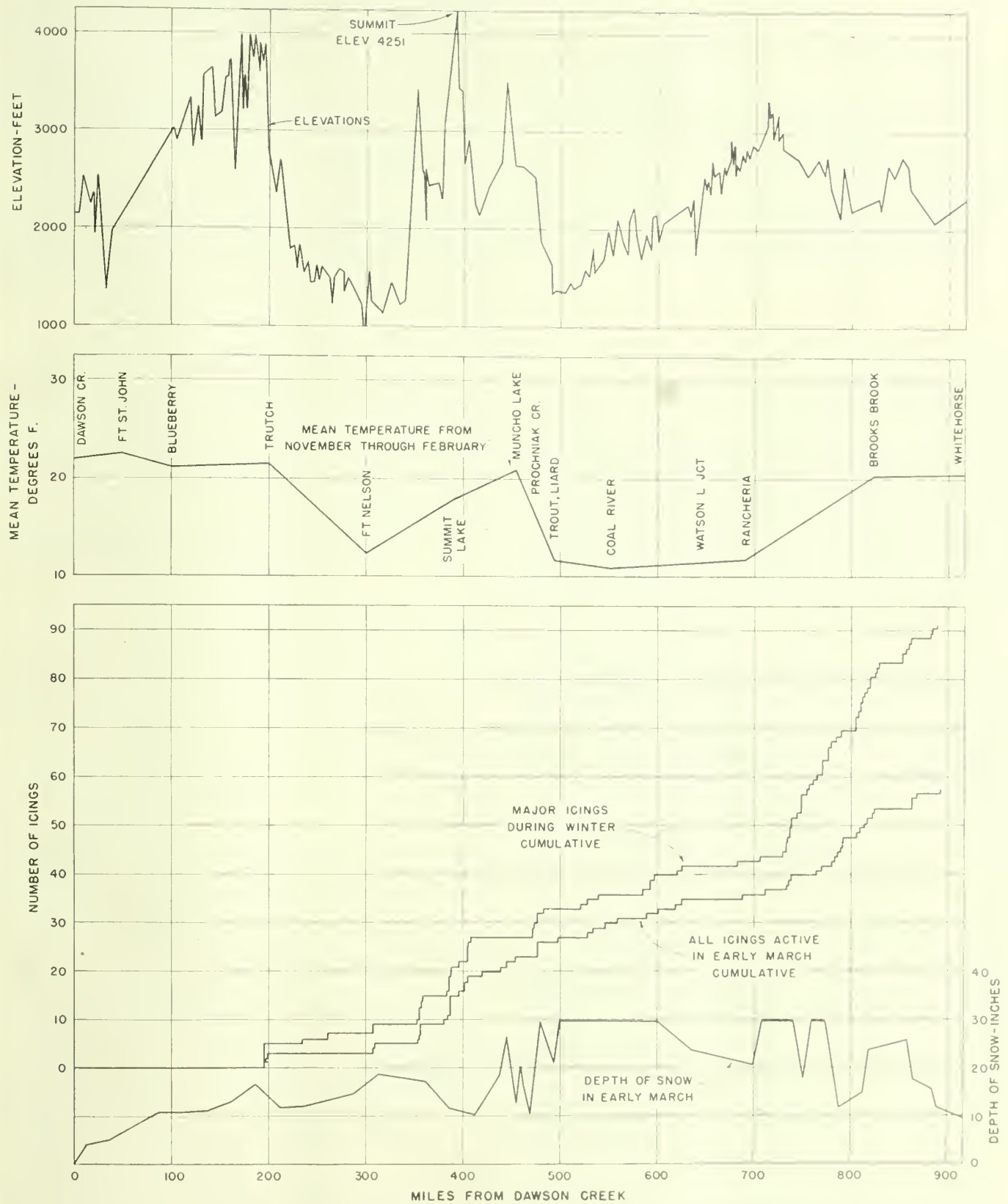


FIGURE 5.—ELEVATION, TEMPERATURE, SNOW DEPTH, AND ICING LOCATIONS BETWEEN DAWSON CREEK AND WHITEHORSE, WINTER OF 1943–44.

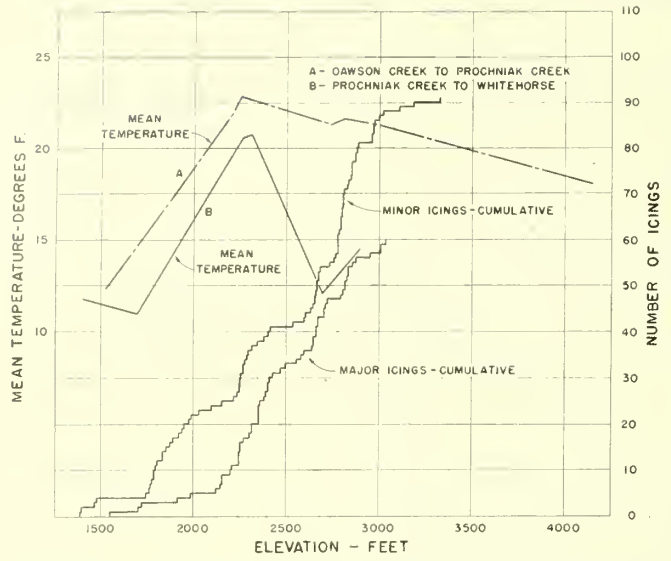


FIGURE 6.—EFFECT OF ELEVATION ON MEAN TEMPERATURE AND ICINGS FROM DAWSON CREEK TO WHITEHORSE FROM NOVEMBER THROUGH MARCH 1943–44.

threatened to invade the road surface. However, this was prevented by throwing up dikes of snow and brush and opening the culverts.

ICING MOST ACTIVE IN JANUARY AND FEBRUARY

Formation of ice was most active in January and February. Figure 1 shows that the minimum mean temperature was reached in January, but there was little recession in icing activity until about a month after the mean minimum temperatures had been passed.

Figure 3 presents data on icings between Prochniak Creek and Whitehorse. It indicates that the major icings in this section continued to increase for about a month after the mean minimum temperature had been passed, but a considerable portion of the minor icings became inactive about 2 months after the onset of the icing season.

DEEP SNOW PREVENTS ICING

Figure 4 shows the number of icing points active at the end of February and the estimated depth of snow cover along the section of highway from Whitehorse to Big Delta. Between Whitehorse and mile 1,163 icings occurred at fairly regular intervals wherever the depth of snow cover was less than 15 inches but did not occur when the snow cover exceeded this amount. There were other factors—topography, type of drainage, temperatures, etc.—which prevented icings at all but four points west of mile 1,163. The area traversed between mile 1,115 and 1,163 is one which might be expected to have heavy icing normally. Several heavy icings on this section were reported the previous winter by Major Hake, but the deep snow cover during the winter of 1943–44 apparently prevented recurrence of the icing.

Figure 5 shows the approximate elevation, mean temperature, observed depth of snow, and cumulative totals of active icings for the section from Dawson Creek

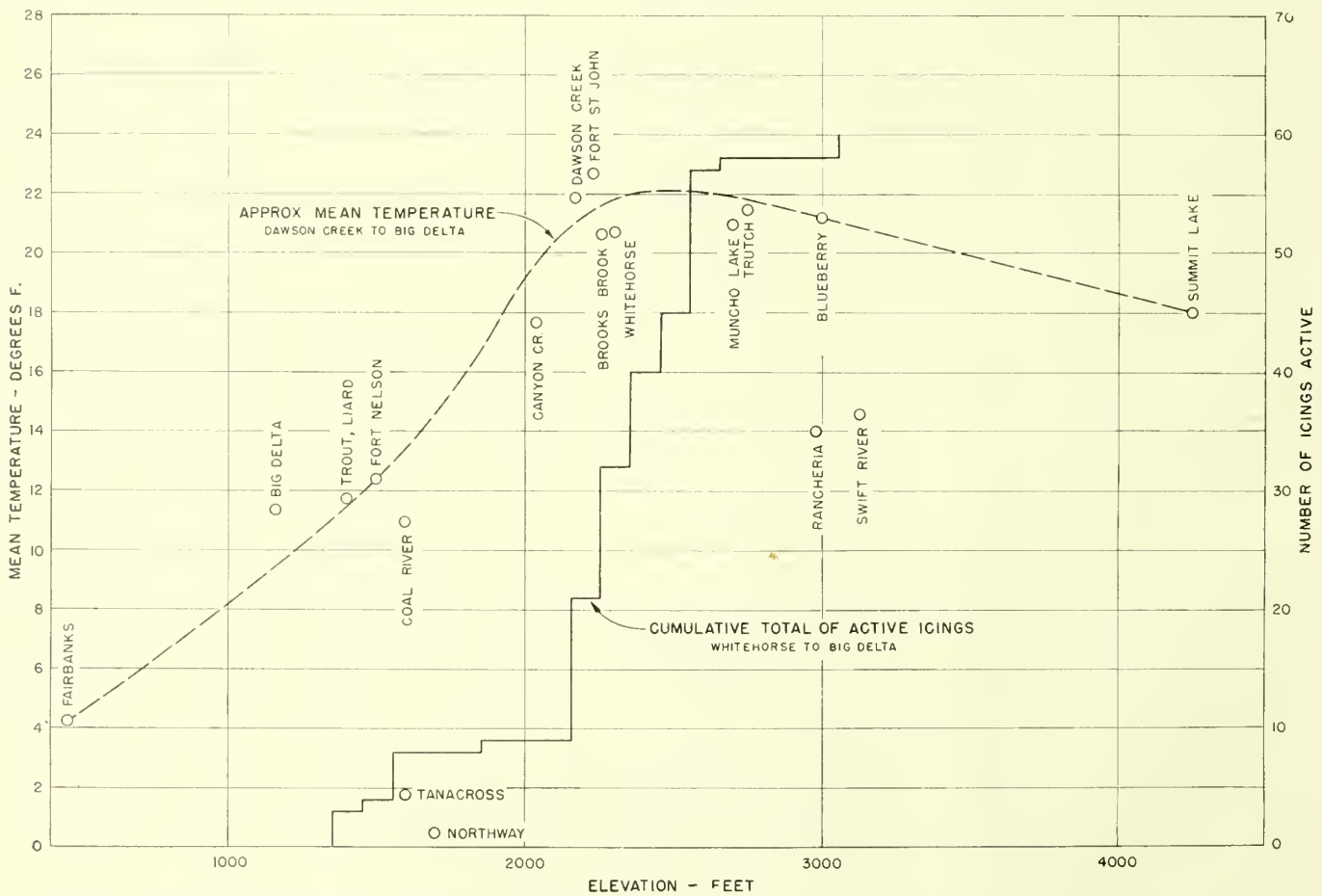


FIGURE 7.—EFFECT OF ELEVATION ON MEAN TEMPERATURE AND ON ICINGS BETWEEN WHITEHORSE AND BIG DELTA, NOVEMBER THROUGH MARCH 1943–44.

A, At This Location Near the Alaska Border the Natural Conditions are Conducive to Icing; B, Deep Snows in the Region Near Kluane Lake Prevented Serious Icing.

to Whitehorse. The observed depth of snow and totals of active icings are for the same period of time in early March when the snow had reached its maximum depth. There appears to be little connection between the elevation and the depth of snow cover, but it will be noted that west of Fort Nelson, the frequency of icing increases as the highway passes over a divide or range of mountains. This is probably largely due to the more adverse drainage conditions from an icing standpoint that occur in such locations.

ELEVATION AFFECTS BOTH THE TEMPERATURE AND EXTENT OF ICING

In figure 6 elevation above sea level is compared with icing location and mean temperature for the Dawson Creek to Whitehorse section. Figure 7 shows similar data for the Whitehorse to Big Delta section. Two conditions are apparent from these figures.

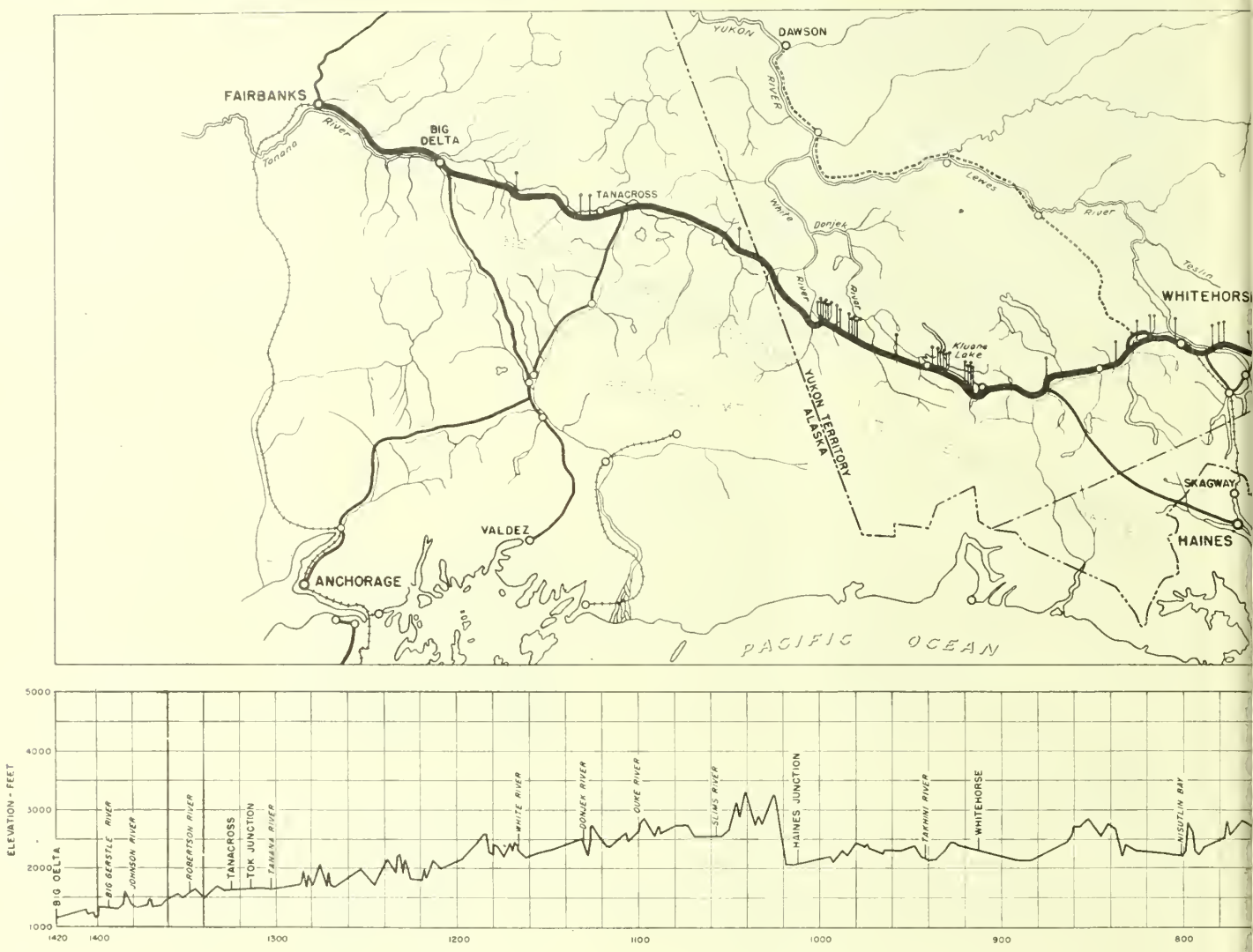
(1) The maximum mean winter temperatures during the winter of 1943–44 occurred at elevations of about 2,300 feet. Mean temperatures were appreciably lower at elevations both above and below this level.

(2) The greatest frequency of active icing was at elevations above 2,100 feet.

The first condition is probably associated with the commonly observed "temperature inversion" in subarctic and polar regions. By means of sounding balloons it has been noted that the temperature during the winter in subarctic regions frequently increases from the ground upwards to some certain elevation above the ground surface, from which point on the temperature again decreases. Over relatively flat ground a stable condition results from this phenomenon which is caused by loss of heat from the snow-covered surface by radiation. In mountainous country the cold air resulting from contact with the cold snow surface being heavier tends to flow to the lowest possible elevations and is replaced at the higher elevations by warmer air. Also, cold polar masses of air commonly move southward and because of their greater density, tend to remain at the lowest possible elevations.

Figure 7 shows the mean winter temperatures at all of the stations reporting weather between Dawson Creek and Fairbanks. Mean temperature at all of the stations except Rancheria, Swift River, Tanacross, and Northway fall quite close to a well-defined curve. These four stations are in river valleys between mountains where the cold masses of air tend to remain, reducing the mean winter temperature.

A, Permafrost Exposed by Grading in Alaska. B, West of Tanacross Where a "Frost Dam" Formed at the Roadway. Hydrostatic Pressure Has Tipped the Trees From Vertical Position. Note the Water on the Surface. C, At the Same Location as B. Hydrostatic Pressure Has Raised the Thick Vegetal Cover at the Right About 6 Feet.



MAP AND PROFILE OF ALASKA HIGHWAY. LOCAT

It may be noted in figures 5 and 7 that the stations below an elevation of 2,100 feet are not in any particular locality or latitude but are scattered over much of the length of the highway.

That the greatest frequency of active icing was found at elevations above 2,100 feet may be explained by the following conditions:

(1) At the higher elevations the topography is more rugged and the water flows intercepted by the highway, although smaller, are more numerous.

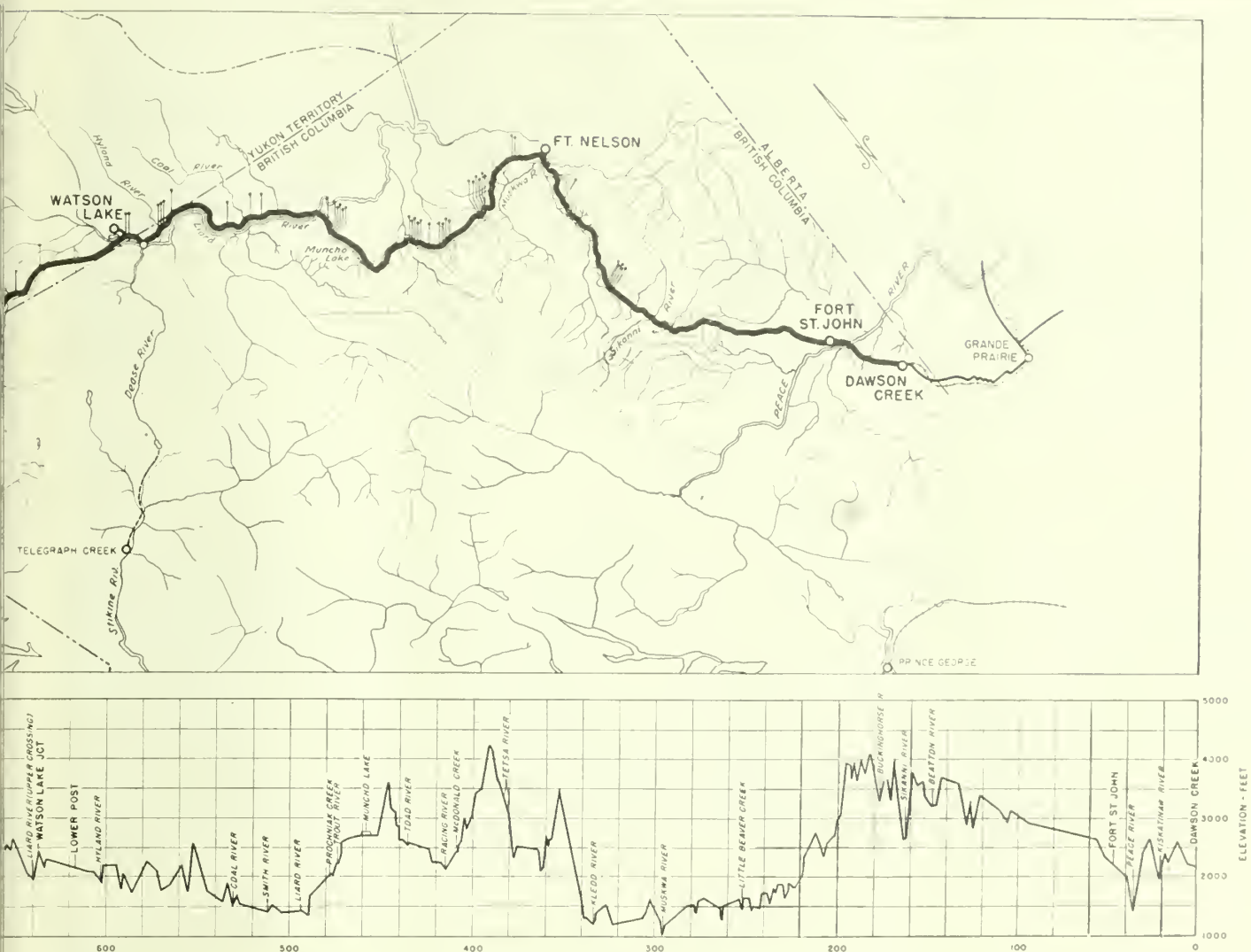
(2) Water flows having steep gradients are more subject to icing and at the higher elevations the watercourses are naturally steeper.

(3) Temperatures during the winter, while averaging higher than at the lower elevations, are apt to fluctuate more widely.

That the greatest frequency of active icing occurred at elevations above 2,100 feet while the lowest mean temperatures were found below that elevation is not in agreement with the general observation that icing activity varies inversely with temperature, the period of greatest icing activity lagging somewhat behind the lowest mean temperatures. It is possible to rationalize these two conditions by considering that icing, to a very large extent, depends upon the flow of water and also upon the temperature. Other things being equal, for maximum icing there must be a maximum flow of water coupled with low temperature. However, when the temperature consistently remains very low, the flow of water is inevitably decreased which in turn decreases the icing. This decrease in flow is bound to lag considerably behind decrease in temperature whereas an increase in icing activity takes place immediately after the temperature drops.

VARIOUS METHODS USED TO PREVENT OR ALLEVIATE ICING

It is impracticable to prevent icing entirely, but there are steps that may be taken in locating, designing, and constructing a highway to reduce or even eliminate the necessity for control of the icing during the winter. Certain locations are most apt to be subject to icing and these locations should be avoided if possible. Generally it will be impossible to avoid all such locations and where they cannot be avoided certain procedures should be adopted to reduce the winter maintenance. This work can be done during initial construction but may also be done by maintenance forces after the location of the most active icing points has been determined by experience. Only a careful study at each icing point will indicate the best methods to use. Records of past



S INDICATED BY LINES ERECTED ALONG ROUTE.

activity at any particular location will help to determine how much preventive work is justified.

Procedures in use on the Alaska Highway or suggested by observations on the highway are:

1. Improvement of drainage channels and structures.
2. Installation of numerous and large drainage structures.
3. Raising the roadway grade.
4. Construction of dikes to confine the flow within certain channels.
5. Construction of subsurface drains.
6. Construction of basins for formation and storage of ice, either at the roadway or some distance upstream.
7. Construction of diversion dikes and ditches.
8. Stripping areas across watercourses and seepage areas to induce icing when winter starts.
9. Construction of dikes to act as storage dams for ice formation.

It has been observed that improvement of drainage channels and structures and raising the grade are always good practices in combating icing. A fundamental rule to follow in constructing drainage ditches for ice control is to make them as deep and as narrow as conditions permit. This will often necessitate placing a culvert well down in the original ground. There should be no break in the channel grade line at the drainage structure. An overflow culvert at higher elevation may be necessary for use if the lower culvert becomes blocked by ice or debris.

Two important advantages are gained in having the drainage trenches as deep and as narrow as possible. Water flowing in the trench receives considerable protection from the close, high banks and vegetation growing on them. Space is provided for a considerable depth of ice formation under which flow channels may be opened later with a steam jet. Considerable protection is thus afforded the water flow by the covering of ice, air space which may occur between the water and the ice, and any snow cover which may fall or be placed on the ice covering. The protection resulting from this combination of conditions was often so effective that the water flow continued under the surface for as much as a month without further attention and in some cases there was a complete cure for the rest of the winter. Ice itself is a relatively poor insulator but it does prevent the loss of heat from underlying water by direct radiation into the atmosphere. Air confined by ice and loose snow supported by it are very effective insulators.

Culverts for cross drainage should be placed at frequent intervals along sections subject to icing, since icing may divert the flow from its normal courses.

Outlet of a Culvert That Is Completely Filled With Ice.

They should be at least double the size necessary to take care of the normal run-off in order to allow for the reduction in effective size by icing. At least part of the culverts should be placed well down in the original ground so that the water flow may be carried in relatively deep and narrow trenches within the limits of construction without a break in the continuity of the gradient.

DIKES AND SUBDRAINS DISCUSSED

A high grade line is often the surest way of avoiding trouble. To be effective there must be high ground along the center line on either side of the point of icing and to be economical the high grade line must be required for only a short distance. When the roadway grade is raised after original construction it will usually be necessary to extend drainage structures. The cost and practical difficulties in the way of such extension will usually determine the feasibility of a higher fill.

Ice forming on the upstream side of the roadway fill constitutes a problem only if it gets high enough to invade the road surface or if a passageway is not opened through this ice to and through the drainage structure in time to take care of the spring run-off. Along the Alaska Highway the spring thaw comes in April—one of the driest months of the year. The run-off is almost entirely from melting snow and ice and is relatively slow. It is necessary to have only a small passageway started under or in the ice. The flow itself will enlarge the passageway as required.

In a wide stream bed with shallow flow—which is the type most subject to icing—the formation of ice is apt to divert the flow of water in new directions. This flow with its resulting ice formation may then reach the highway at locations where no drainage structures, channels, or provision for ice storage or ice control have been provided. At such places, dikes can be effectively used to direct the flow within certain channels. Usually it is best to construct the dikes in the fall before freeze-up but after the water flow has subsided to winter normal.

Dikes can be thrown up in a short time with a bulldozer, but are apt to be washed out by the next summer's high water, and may need to be replaced each fall. A more permanent dike can be constructed by riprapping the face of the dike. The channel provided should be as wide as possible to give space for the formation and storage of a large volume of ice before the dike is overtopped. A relatively deep and narrow channel may be constructed within the diked area to direct the flow to the drainage structure until ice begins to cover the entire diked area.

Springs or seeps in cut banks often can be effectively intercepted by porous subdrains if permafrost does not exist. The drain must be below the depth of frost penetration. Five or six feet will ordinarily be adequate in the region of the Alaska Highway. The drain must have an outlet in a protected area. Protection can be afforded by covering with moss and vegetative material common in wet areas along the Alaska Highway and may be further increased by snow thrown off the roadway in maintenance operations. An exposed subdrain outlet will quickly freeze and the water flow may be forced to seek a new outlet in or above the highway. Subdrainage, if successful, offers a permanent cure of icing but requires careful study for adaption to a particular location.

Icing basins immediately upstream from the roadway serve somewhat the same purpose as a high grade line. If the flow is small or active for only a short time during the winter, it may be possible to provide a basin that will hold all of the ice forming during the winter.

In some places, it may be possible to construct dikes to divert the water to some new channel or to an area where the formation of ice will not affect the roadway. This was done at two places in the Rocky Mountain region west of Fort Nelson during construction of the highway in 1943. In both cases the flow was effectively diverted to strike a main stream channel without crossing the highway and icing on the highway was entirely eliminated.

INDUCED ICING EFFECTIVE IN CERTAIN LOCATIONS

Under certain conditions icing may be induced at some point up hill from the roadway by exposure that causes some or all of the water flow to freeze at that point and correspondingly reduces the icing at the roadway. Icing is usually induced by stripping or compacting the vegetation and snow cover so that the insulation normally provided is largely destroyed. The ground itself may readily freeze upon exposure forming a "frost dam" and forcing an underground flow or seepage to the surface where it will form into ice. For such a method to be effective in preventing the formation of ice at the roadway the following conditions must exist:

(1) The flow of water must be near the surface.

(2) The flow of water must be light or the space available for ice storage must be large. A flow of 6.9 gallons per minute will cover an acre of ground with 1 foot of ice in a month's time.

(3) Conditions must be such that the flow will not merely be diverted around the initial ice formation and strike the highway at a new place to cause icing there.

(4) Snow cover at the induced-icing area must be removed or compacted whenever the accumulation prevents freezing.

An induced-icing area can be effectively combined with dikes and ditches to accomplish the following results:

Ditches can be constructed to drain wet areas during the summer and fall and thus decrease the flow of water

A, Icing Started at Roadway Early in the Winter. Stripping an Area Across the Watercourse at This Point Induced Icing and Protected the Roadway for the Remainder of the Winter. B, An Induced-Icing Area Made With a Bulldozer. C, A Building Placed Over the Culvert Inlet to Increase Heating Efficiency. D, Storage of Ice and Water at an Induced-Icing Area.

during the winter. The ditches should be made narrow and deep to carry winter flow with a minimum of freezing.

Material excavated from the ditches can be used to form a dike behind which the ice can be stored, thus increasing the ice storage area.

The ditches and dikes can be so arranged as to insure that such water as does not freeze at the induced-icing area will reach the roadway at a point where it can be effectively handled.

Frost will readily penetrate the area exposed by the ditch and dike construction, creating a "frost dam" and causing subsurface water flows to emerge and freeze.

An area on which icing is to be induced need not be level. Thin films of water upon freezing will form masses of ice with a surface conforming to that of the area over which the flow occurs. The ice formed has the appearance of a heavy, viscous liquid in motion. A dam or dike to hold ice need not be watertight; brush or similar obstructions will effectively restrain the flow to form ice. While ice will form to follow the slope of the ground in the direction of normal flow, it will fan out laterally to an elevation about equal to or in some cases greater than the elevation of the ice surface at the middle of the watercourse.

Icing induced away from the highway must generally be watched and controlled or its effectiveness will soon be lost. If a "frost dam" completely seals off the subsurface flow further trouble is prevented. This occurred at several places along the Alaska Highway.

Many of the preventive or alleviative measures adopted resulted in the formation of large masses of ice upstream from the highway. Some may wonder whether the large volumes of ice constituted a hazard at the time of the spring thaw. Actually, the masses of ice melted slowly and did not cause any difficulty.

CONTROL MAINTENANCE NECESSARY IN ADDITION TO PREVENTIVE MEASURES

Where all reasonable ice-preventive measures have been taken in the location, design, construction, and summer maintenance of the highway it may still happen that, as the winter progresses, ice may threaten to invade the road surface. Some method of direct control must then be applied. The primary purpose will be to keep ice from forming on the road surface where it would interfere with the normal use of the highway. Three general methods were followed in control of the icing on the Alaska Highway during the winter of 1943–44:

(1) Heating the flowing water by artificial means to prevent its freezing. This was done in locations near the highway.

(2) Periodic removal of the ice formed, usually by mechanical means, but sometimes by melting the ice using artificial heat.

(3) Carrying the flow in channels in the ice. These channels required periodic maintenance, the amount depending upon the degree of protection given them.

On induced-icing areas some control maintenance is necessary. Snow must be removed or compacted from time to time and the dikes raised or new ones constructed. Maintenance of induced-icing areas is to cause ice to form rather than prevent its formation.

The methods used on the Alaska Highway during the winter of 1943–44 were selected largely by superintendents and maintenance men. Various ideas were developed and an opportunity afforded to study and compare them. The control methods used are described in more detail in the following paragraphs.

FREEZING PREVENTED BY HEATING

Heating can effectively prevent the formation of ice at the roadway by warming the flow of water. All heating methods are expensive as equipment must be obtained and its use requires frequent attention as well as considerable fuel. With the heating equipment originally available on the Alaska Highway it was found difficult to concentrate the heat where it would be most effective and most of it was dissipated into the atmosphere. To concentrate the heat in the water where it would be most effective, the Army maintenance organization designed and built an oil burner shown in the accompanying illustration. Oil is fed from the tank on top of the heater through a small tube which passes through a larger draft tube nearly to the bottom of the oil drum where combustion takes place.

Since icing inside of culverts was one of the chief difficulties, methods intended to prevent freezing there were devised. Wood and oil fires were kept burning in open-end oil drums standing in the water at the inlet end of culverts. While most of the heat developed by these fires was lost to the atmosphere, some of it did warm the flow of water, usually enough to prevent icing in the culvert. Heaters were also placed at the outlet ends to prevent icing there and consequent blocking of the culvert. To give greater heating efficiency small houses were frequently built around the stoves at the ends of the culverts. The oil stoves burned from 10 to 20 gallons of fuel oil per day under extreme conditions. In some cases small oil-burning flares were placed inside of the culverts. The heat thus generated was more efficiently used, particularly when the ends of the pipe were almost entirely covered to prevent loss of heat to the outside atmosphere. This type of heater consumed only 3 to 4 gallons of fuel oil per day.

In many place the heating devices were spaced at intervals along trenches cut in the ice. The heat developed at each unit was sufficient to keep the water flowing to the next fire and so on until it passed through the drainage structure and beyond the roadway. Thus a number of heating units might be set up both upstream and down from the roadway to control one icing. In periods of extreme cold, fires were required at intervals of about 50 feet along any open trench where it was desired to maintain the water flow.

Under the most adverse temperature conditions, icing will occur in spite of all such heating methods. At an icing location the points where water emerges may shift frequently and with each change the entire heating set-up must be changed. This is a troublesome and expensive task and a frequent check of each heating set-up is required.

Advantages of this method of icing control are: The necessary equipment is simple and may be readily made or repaired with the facilities available at each maintenance camp; skilled operators are not required; and control of icing does not depend upon the continued operation of any one unit of equipment. The disadvantages are: The method is expensive both in fuel and labor, it is inefficient, and in periods of extreme low temperature does not prevent icing.

ICE REMOVED WITH TRACTOR RIPPERS AND GRADERS

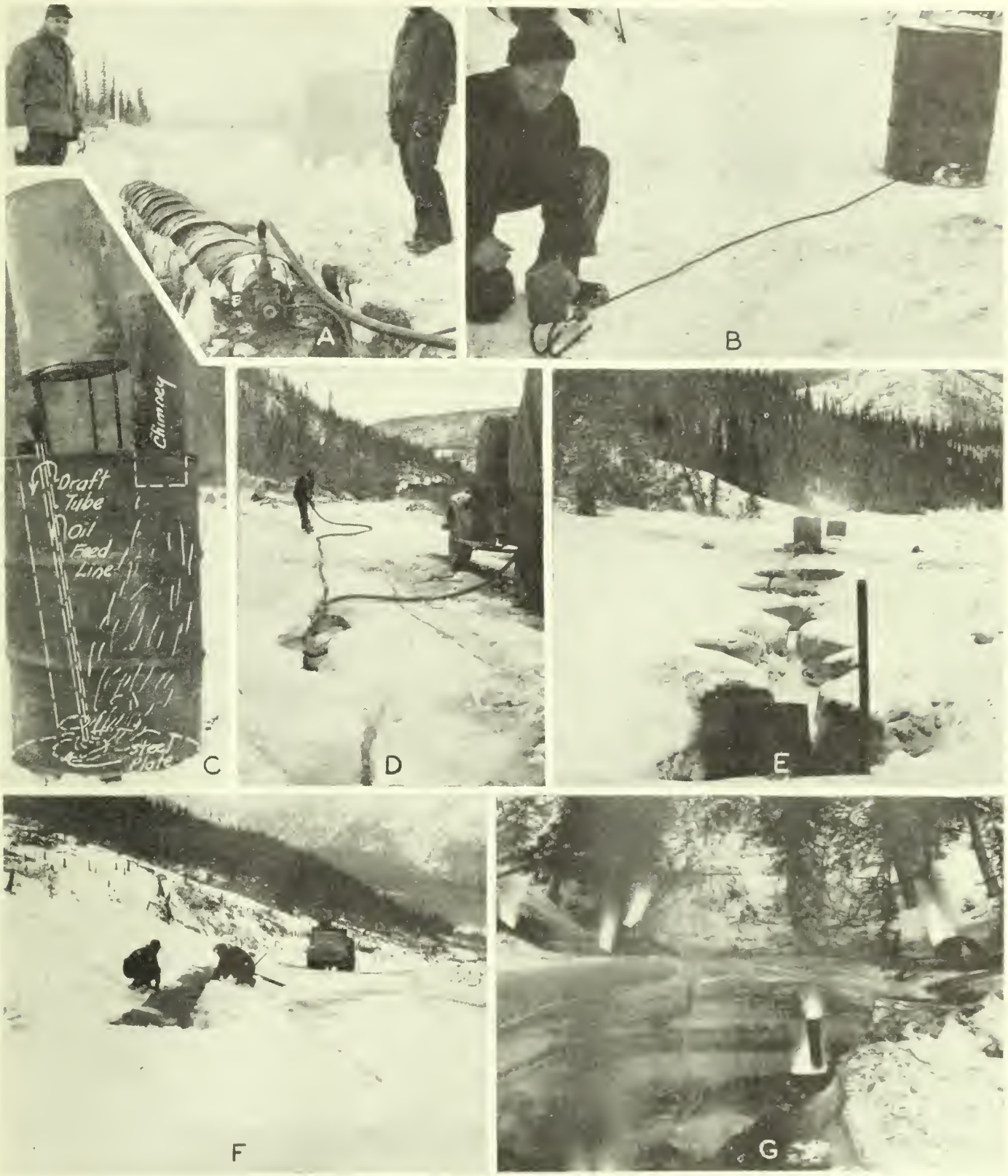
At some places the ice was removed periodically with tractor rippers, blade graders, and motor patrols; by blasting; and by thawing with large oil-burning torches or wood fires. Where these methods were used no attempt was made to control the formation of ice. It was merely removed whenever the formation began to encroach upon the road surface. No advantage was taken of the natural laws governing the formation of ice.

Tractor rippers, blade graders, and motor patrols were used regularly to remove ice on only two sections. One was the Steamboat Mountain section west of Fort Nelson and the other was a 2-mile section with rock hillsides between Fort St. John and Fort Nelson. On this last section blasting was also done to break up the ice along the inside road ditch. On both sections the roadway was constructed as a cut and fill on a fairly steep hillside and with a relatively narrow ditch on the inside. Ice formed on the back slopes and in the roadside ditches from seepage and spring water. As the winter advanced, the flow of water gradually spread out in both directions from the original source by contour spreading along the back slopes so that eventually the ice formed on the back slopes and in the ditches for considerable distances along the highway.

Whenever formation of ice on the roadway impended, the ice was broken out along the ditch line by a tractor-drawn one-tooth ripper and bladed across the roadway and over the shoulder with a tractor-drawn grader or motor patrol. It was necessary to repeat this operation every day or two during the colder part of the winter. Even then removal did not quite keep up with the ice formation but a travelable surface was maintained. Actually the highway surface was gradually built up and widened with broken ice fragments cemented together with ice films. This material was not slippery, however, and caused no difficulty as long as it remained frozen. It seems quite possible that such an accumulation of ice might create a hazardous condition at the time of the spring thaw but no particular difficulty was reported on either of the sections where the method was used.

While this system is less efficient and more costly than other methods, there are occasions when it must be resorted to. If icing occurs continuously on a section quite close to camp and is rather slow in forming so that it can be easily controlled by the occasional removal of ice, and if there is other work in the area to justify the maintenance of heavy equipment, then such procedure may be justified.

There is little to recommend the method for general use. A large crawler tractor is the only machine with enough power and traction to pull or push the equipment necessary to break up the ice. Such a unit is expensive to operate, particularly under such extreme weather conditions. If the ice formations are spread out over a considerable length of highway, much time will be lost in traveling from one to another or else several complete outfits must be maintained. Unless



A, Thawing a Channel With a "Flame Thrower" and Inverted Halves of Oil Drums; B, Oil-Burning Torch Used Inside Culverts; C, Oil-Burning Heater; D, Opening a Trench With a Steam Jet, Steam Generator is Mounted on Truck; E, Channel Kept Open by Wood Fires in Oil Drums; F, Placing Tar Paper Over a Trench in the Ice; and G, Oil Fires in Drums.

THE STEAM GENERATOR IN ACTION.

the tractor is placed in a heated garage at night or operated 24 hours a day, it may be difficult to start it and considerable time may be lost. Ice forms in many places inaccessible to heavy equipment. In culverts and inlet and outlet ditches it must be removed by other means, so that other equipment must be available.

BLASTING OF ICE HAS DISADVANTAGES

Blasting was resorted to in certain areas to loosen ice in roadside ditches. Ice broken by blasting is not entirely removed from its original position and hand or machine work may be necessary to supplement the blasting. As the ditches again fill with ice the blasting operation must be repeated. At some places the ice broken by blasting was not removed from the ditch and later was covered with snow plowed from the roadway surface. This appeared to be good practice as the small amount of seepage water readily percolated between the broken fragments of ice and was prevented from freezing by the insulating covering of snow.

The blasting method is probably not as expensive as the tractor-ripper, blade-grader, and motor-patrol method and may be used in places inaccessible to heavy equipment. It should not be used to open culverts or in the vicinity of telephone lines, camps, or other facilities. After a few unfortunate experiences with broken telephone wires blasting as a method of ice control was largely discontinued along the Alaska Highway. Blasting is sometimes effective in starting induced icing or in opening subsurface channels for the passage of seepage water.

NARROW CHANNELS THAWED WITH STEAM GENERATOR GIVE EFFECTIVE CONTROL

Occasionally results similar to those with the tractor-ripper and blasting methods were obtained by actual thawing of the ice. This method is expensive and has little to recommend it. Heat is supplied either by oil-burning torches or by wood fires. The flame thrown by torches was usually confined under half sections of oil drums or culvert pipe for more efficient heat direction against the ice to be thawed. While the large oil-burning torch or "flame thrower" as it was called thaws the ice quite rapidly, it requires about 6 gallons of oil per hour. It cannot safely be used in wooden drainage structures and its general use is not recommended.

Carrying water in either protected or open channels made with portable steam generators such as were made available along the highway during the latter part of the winter was found to be a useful method. Other methods were also used to construct open channels in the ice. The steam jet is well adapted to cutting a confined channel so that a minimum of water surface will be exposed to the atmosphere. Flow in the channels may be facilitated by supplemental heating, as previously described, or the channels may be reopened at intervals as the icing development necessitates.

This method, combined with channel protection appeared to offer the greatest possibilities of control of icing, after it actually begins, of any method observed along the highway. Full advantage may be taken of the insulating properties of ice, snow, and air. The ice must first be allowed to form to a depth of 2 feet or more to provide a medium through which drainage channels may be cut. This ice will insulate the flow, and will support further insulation such as paper, boards, brush, or snow falling directly on the covering or thrown off the road surface by maintenance forces. If conditions are such that the required depth of ice cannot be permitted to form without danger of icing on the road surface, then much more frequent attention will be required. Grade raises, icing basins, and deep and narrow drainage trenches permit the formation of ice to such depth that effective channels may be opened through or under the ice by a steam jet.

If a steam jet is used for cutting passageways under the ice, it is best to start operations at the culvert or drainage structure outlet. As a channel is formed under the ice in the drainage structure, the water from the condensed steam and melted ice, will enlarge the opening already formed and will also, in most cases, find an outlet for itself under the vegetation and snow downstream. It is good practice to leave this downstream cover undisturbed, and allow the outflow of water to form its own subsurface drainageway.

After an opening has been made under the ice in the drainage structure, a channel can be formed under the ice from the culvert inlet to the apparent source of the flow. This can be done by pushing the steam jet down through the ice at intervals of about 3 feet. It may take some probing with the steam jet to locate the source of the water but when this is done a considerable volume of water is likely to be released and further icing activity stopped for some time. At a number of places the method was so successful as to give a complete cure for the rest of the winter.

Trenches jetted with steam are definitely better than those made by other methods. Blasting forms wide, irregular trenches that refill rapidly. Trenches may be cut by hand but the method is slow and it is difficult to cut to the required depth. However, the Alaska Road Commission controls icing on some of its roads entirely by hand trenching.

The equipment most used for steam jet operation was a quick-acting oil-burning steam generator, which was usually mounted in an enclosed truck. The factory price of a unit was about $3,200. The generators were very effective, producing a large volume of steam within a very short time and were readily moved from one icing location to another.

The apparatus was somewhat complicated and required careful operation for continued trouble-free use. Apparently it has been designed primarily for stationary use as it was susceptible to damage by rapid movement over rough surfaces.

A number of these units were placed on the highway. Control methods developed with their use proved so

(Continued on p. 82)

AN EMPIRICAL METHOD OF INTERPRETATION OF EARTH RESISTIVITY MEASUREMENTS[1]

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by R. WOODWARD MOORE, Associate Civil Engineer

MANY PAPERS have discussed the interpretation of data obtained from earth resistivity tests when using the four-terminal method of electrode spacing developed by Wenner (*9*).[2] Most of these have dealt with theoretical analyses for two-layer and three-layer formations. In some instances, sets of reference curves have been presented for use in analyzing field data to determine the depth to the first and possibly the second horizon below the earth's surface. Although practically all of these theoretical methods of analysis have appeared to have particular merit and, in some cases, have been successfully used in practice, they have been found to be of little value in those cases where the local conditions surrounding the test failed to conform to those assumed in the theory.

In certain fields, particularly in civil engineering, relatively shallow explorations are often required and geophysical methods of test must compete with the direct methods of exploration ordinarily used. Only when it can be demonstrated that geophysical methods of test can materially reduce the time and cost of a given exploration project will the civil engineer abandon direct methods in their favor.

Empirical methods of analyzing earth resistivity data have been used in many instances in the past. Such methods have been used for a number of years by the Public Roads Administration in research on application of the resistivity test in the field of highway construction. Prior to 1940 the empirical relation proposed by Gish and Rooney (*2*) was generally used in the study of field data. In this method of analysis, the apparent resistivities measured for a succession of gradually increased electrode spacings are plotted as ordinates and the corresponding electrode spacings as abscissas. There appears to exist an empirical relation that the "effective depth" of current flow approximates the value of the electrode spacing used. If, within this "effective depth", an underlying formation with a specific resistance materially different from that of the surface layer is encountered an inflection will presumably be found in the apparent resistivity-electrode spacing curve.

Gish and Rooney found that where such an inflection appeared the value of the electrode spacing at the point of inflection could be interpreted as the approximate thickness of the surface layer. Where a low resistivity material is underlain by one of a higher resistivity a U-shaped curve is often obtained and in such cases the electrode spacing for the low point of the curve is usually assumed to indicate the depth to the underlying material.

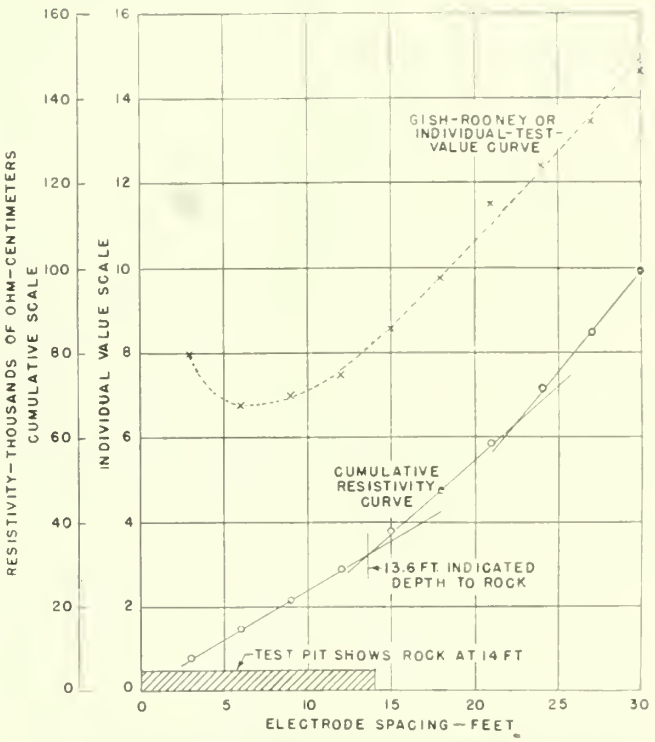


FIGURE 1.—TYPICAL RESISTIVITY DATA AND METHOD OF ANALYSIS USING THE CUMULATIVE RESISTIVITY CURVE.

TABLE 1.—*Data on resistivity and electrode spacing used in plotting figure 1. Data obtained where a test pit showed 14 feet of clay overlying hard rock*

Electrode spacing	Apparent resistivity	Cumulative resistivity
Feet	*Ohm-centimeters*	*Ohm-centimeters*
3	8,000	8,000
6	6,800	14,800
9	7,050	21,850
12	7,510	29,360
15	8,600	37,960
18	9,800	47,760
21	11,550	59,310
24	12,430	71,740
27	13,500	85,240
30	14,650	99,890

Often in shallow work abrupt breaks in the apparent resistivity-electrode spacing curves have been found which, when interpreted in this manner, give a reasonably accurate indication of the position of the interface between the surface materials and the underlying material in simple two-layer formations. There are other instances, however, in which resistivity depth tests in two-layer or multilayer formations yield resistivity-electrode spacing curves that are smoothly rounded

[1] Paper presented at the February 1944 meeting of the American Institute of Mining and Metallurgical Engineers and published by them as Technical Publication 1743.

[2] Italic numbers in parentheses refer to the bibliography at the end of this article.

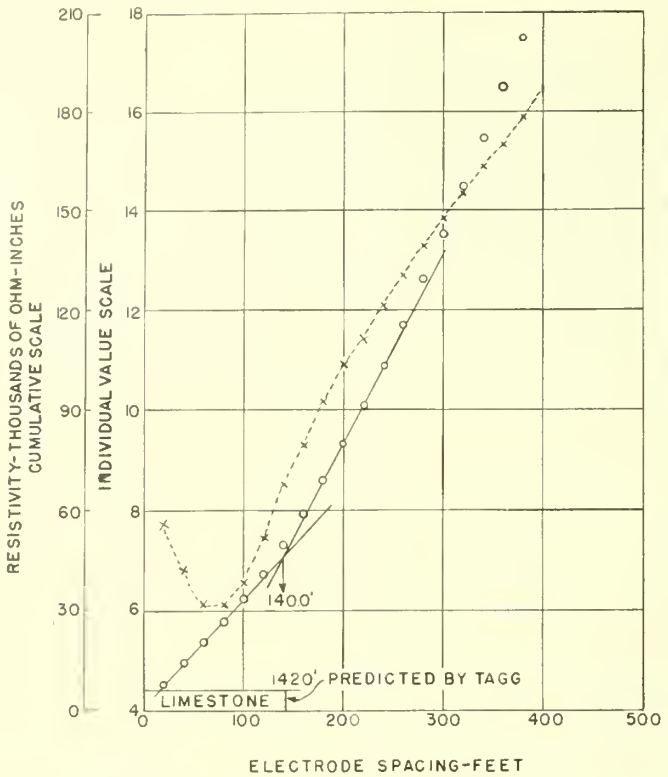


FIGURE 2.—REPRODUCTION OF CURVE FOR "STATION A" FROM MEASUREMENTS BY G. F. TAGG.

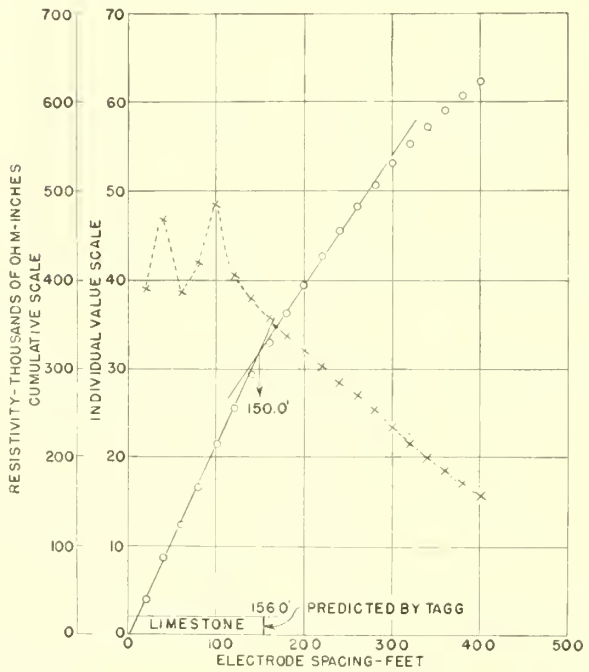


FIGURE 3.—REPRODUCTION OF CURVE FOR "STATION B" FROM MEASUREMENTS BY G. F. TAGG.

without marked inflections or other indication of the position of the various strata. No satisfactory empirical method of analysis of such curves is available and such theoretical methods as have been proposed are both uncertain and time consuming.

There follows a discussion of an empirical method of analysis developed in the resistivity research of the Public Roads Administration. When applied to test

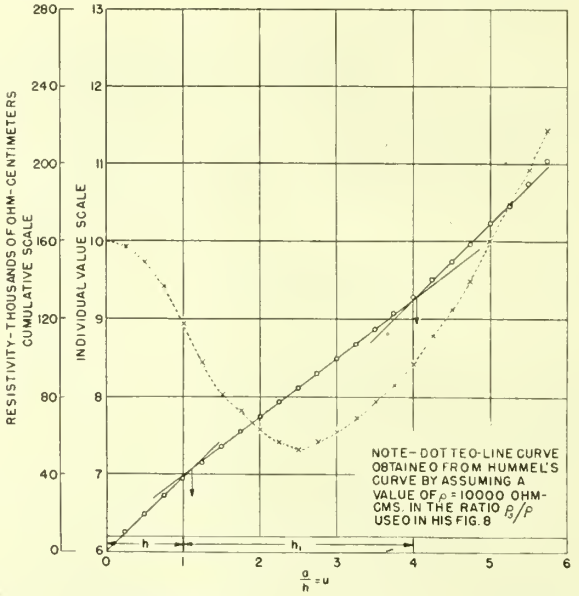

RESISTIVITY-THOUSANDS OF OHM-CENTIMETERS
CUMULATIVE SCALE
INDIVIDUAL VALUE SCALE
NOTE-DOTTED-LINE CURVE OBTAINED FROM HUMMEL'S CURVE BY ASSUMING A VALUE OF ρ = 10000 OHM-CMS. IN THE RATIO ρ/ρ USED IN HIS FIG. 8
h
h1
a/h = u

FIGURE 4.—HUMMEL'S CURVE OF APPARENT RESISTIVITY FOR CASE OF $H_1 = 3H$ AND RATIO $\rho_0 : \rho : \rho_1 : \rho_2 = \infty : 1 : 0.5 : \infty$.

data of the type described above from various sources and over a wide range of field conditions, it seems to offer definitely better correlations than other methods.

PROPOSED METHOD OF ANALYSIS DESCRIBED

In the proposed method of analysis the data are obtained in the field using the Wenner four-electrode configuration, and a conventional Gish-Rooney or apparent resistivity-electrode spacing curve is prepared. The Gish-Rooney curve is used for whatever indication it may give of subsurface conditions at the point of test. The data for the curve are replotted on the same sheet in the form of a cumulative resistivity-electrode spacing curve. By reason of a greatly reduced ordinate scale and the effect of successive summations of individual resistivity values this plotting of the data tends to minimize the effect of a single resistivity value and thus eliminate purely local effects caused by surface anomalies or any peculiarity of a particular setting of the electrodes.

In obtaining the field data for plotting in this manner an initial electrode spacing of some convenient value, say 3 feet for shallow work, is chosen arbitrarily and the electrode spacing is then increased regularly by increments of 3 feet for each successive determination. The initial value of apparent resistivity is plotted as the initial ordinate of the cumulative curve. Each subsequent value of apparent resistivity is added to the sum of all preceding resistivity values and each total thus obtained is plotted as the ordinate of another point for the cumulative curve. Use of regularly increased electrode spacings, for example, 3 feet, 6 feet, and 9 feet, may suggest that approximately a straight line with a given slope should be obtained so long as the "effective depth" of current flow remains primarily within the surface layer and this layer consists of a relatively homogeneous material. In practice, however, where the surface material is relatively shallow and the soil layer is not perfectly homogeneous, the plotted data frequently will take the form of a generally smooth line with gentle curvature rather than a straight line. Probably this is due to a gradual change of resistivity with depth and with soil variation.

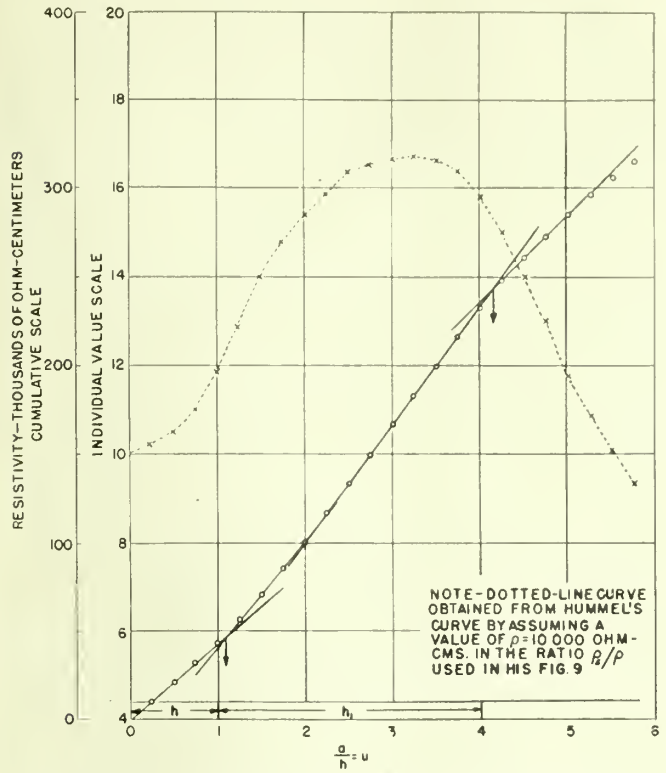

RESISTIVITY—THOUSANDS OF OHM-CENTIMETERS
CUMULATIVE SCALE
INDIVIDUAL VALUE SCALE
NOTE-DOTTED-LINE CURVE OBTAINED FROM HUMMEL'S CURVE BY ASSUMING A VALUE OF ρ=10 000 OHM-CMS. IN THE RATIO ρ2/ρ USED IN HIS FIG. 9
h
h1
a/h = u

FIGURE 5.—HUMMEL'S CURVE OF APPARENT RESISTIVITY FOR CASE OF $H_1=3H$ AND RATIO $\rho_0:\ \rho:\ \rho_1:\ \rho_2=\infty:\ 1:\ 3:\ 0$.

It has been found that as the electrode spacing approaches and passes a value corresponding approximately to the depth of the surface layer the plotted cumulative curve usually tends to change direction, the new slope depending upon the relative resistivities of the two layers of material. From the electrical principles involved this slope should increase if the lower formation possesses a higher resistivity than the surface layer and conversely should decrease if the underlying formation is the more conductive. It has been found that lines drawn tangent to the cumulative curve and intersecting in the region where the change in slope occurs give a good approximation of the depth to the interface between the two materials if the point of intersection of the tangents is projected to the horizontal or dimensional axis.

Figure 1 illustrates the method applied to a typical case. The data used in plotting the curves are given in table 1. The values of electrode spacing and of apparent resistivity in columns 1 and 2, respectively, were obtained in field tests in the vicinity of Washington, D. C., where there existed a simple two-layer formation consisting of clay underlain by rock.

In figure 1 the Gish-Rooney or individual-test-value curve is shown by crosses connected by a dotted-line curve and the cumulative resistivity curve by plotted circles. For clarity the curve connecting the circles has been omitted. This same method of presenting the data is employed in each of the subsequent figures and for simplicity the identification of the curves is not repeated.

Referring to figure 1, the presence of the rock formation of high resistance at the relatively shallow depth of 14 feet affects strongly the measured apparent resistivity beyond an electrode spacing of about 10 feet and for this reason the plotted values of cumulative resistivity continue to show a rather marked degree of curvature beyond what might be termed the "critical point" on the curve. The trend of the Gish-Rooney or individual-test-value curve is used to indicate the probable "critical point" which on this curve appears to be at an electrode spacing of 10 or 12 feet. Guided by the indications of this curve and such related data as may be available from test pits or drill holes in the general area, the additional tangent intersections beyond the "critical point" may or may not be disregarded.

Admittedly, this procedure is empirical in every sense, but the results obtained in the analysis of data from many tests under various field conditions have been encouraging. In addition to the use of this method in the analysis of data obtained in field tests of the Public Roads Administration, the literature has been reviewed and resistivity curves presented by various authors have been replotted and analyzed by the method wherever the necessary depth data were given. The results obtained appear to substantiate the conclusions reached in analyzing the test data of the Public Roads Administration.

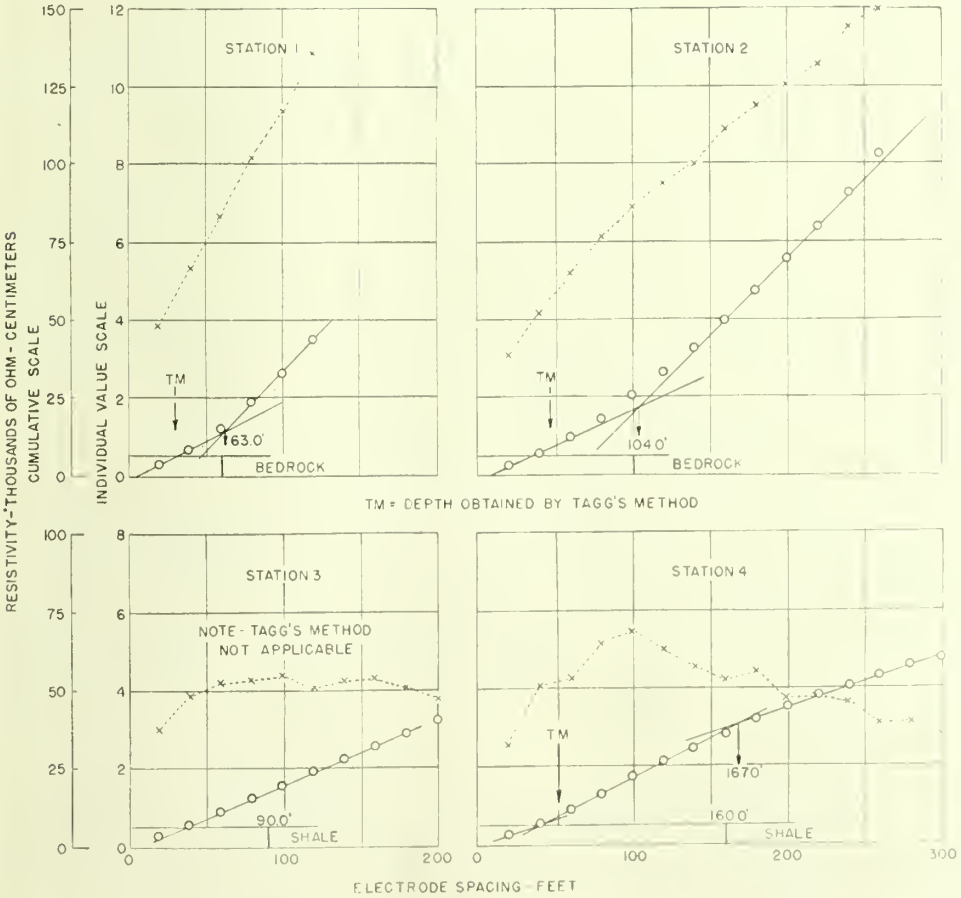


FIGURE 6.—CURVES PLOTTED FROM FIELD DATA OF M. KING HUBBERT.

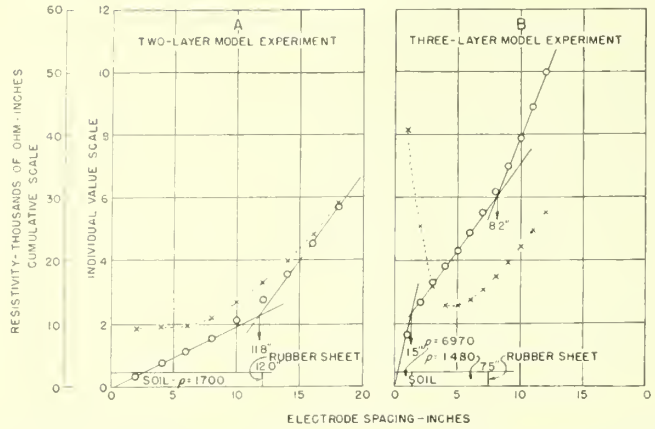


FIGURE 7.—CUMULATIVE RESISTIVITY CURVE METHOD OF ANALYSIS APPLIED TO LABORATORY DATA OBTAINED BY R. J. WATSON.

CUMULATIVE CURVE METHOD APPLIED TO DATA REPORTED BY VARIOUS INVESTIGATORS

Figure 2 shows Tagg's (*7*) classical curve for his "Station A," representing typical field data to which he applied his method of analysis, together with the points that result when the data are replotted in the manner previously described. Figure 3 shows a similar treatment of Tagg's "Station B." The cumulative values in figures 2 and 3 indicate depths that closely approximate those obtained by Tagg with a rather laborious method of analysis. In connection with these curves and others that are to be presented, it should be pointed out that frequently it is rather difficult to read accurately values of coordinates from the necessarily small figures found in the published papers when taking data for replotting.

Figures 4 and 5 show data from Hummel's (*4*) curves of apparent resistivity for two theoretical cases [3] in which two surface layers of differing resistivity and thickness are underlain by, in one case, a layer having infinitely high resistivity and, in the other case, a layer having infinitely high conductivity. The ordinate v of Hummel's figures is given as the ratio ρ_s/ρ in which ρ is the true resistivity of the upper layer of the ground and ρ_s is the average or apparent resistivity resulting from the influence of the underlying layers on the values of resistivity as measured at the ground surface. In plotting the dotted-line curves in figures 4 and 5 a value of 10,000 ohm-centimeters was assumed for ρ and applied to the ratios ρ_s/ρ taken from Hummel's curves. This resulted in conventional apparent resistivity-electrode spacing curves from which the data for plotting the cumulative curves were obtained. The positions of the interfaces as determined from the cumulative curve appear to agree with those assumed in the theoretical treatment rather closely despite the fact that a three-layer condition was involved in both instances.

The curves shown in figure 6 were plotted from data which were presented in a paper by Hubbert (*3*). The data for his stations 1, 2, and 4 when plotted using the cumulative values of resistivity give depth indications that check the known conditions reasonably well. As stated by Hubbert, Tagg's method of analysis, although giving good intersection zones for groups of curves plotted for each of his stations 1, 2, and 4, gives depth indications that are in error by 50, 54, and 70 percent, respectively. The individual-test-value or Gish-Rooney curves for these stations were of little value in determining the depths of interfaces. The data for sta-

[3] Hummel's figures 8 and 9.

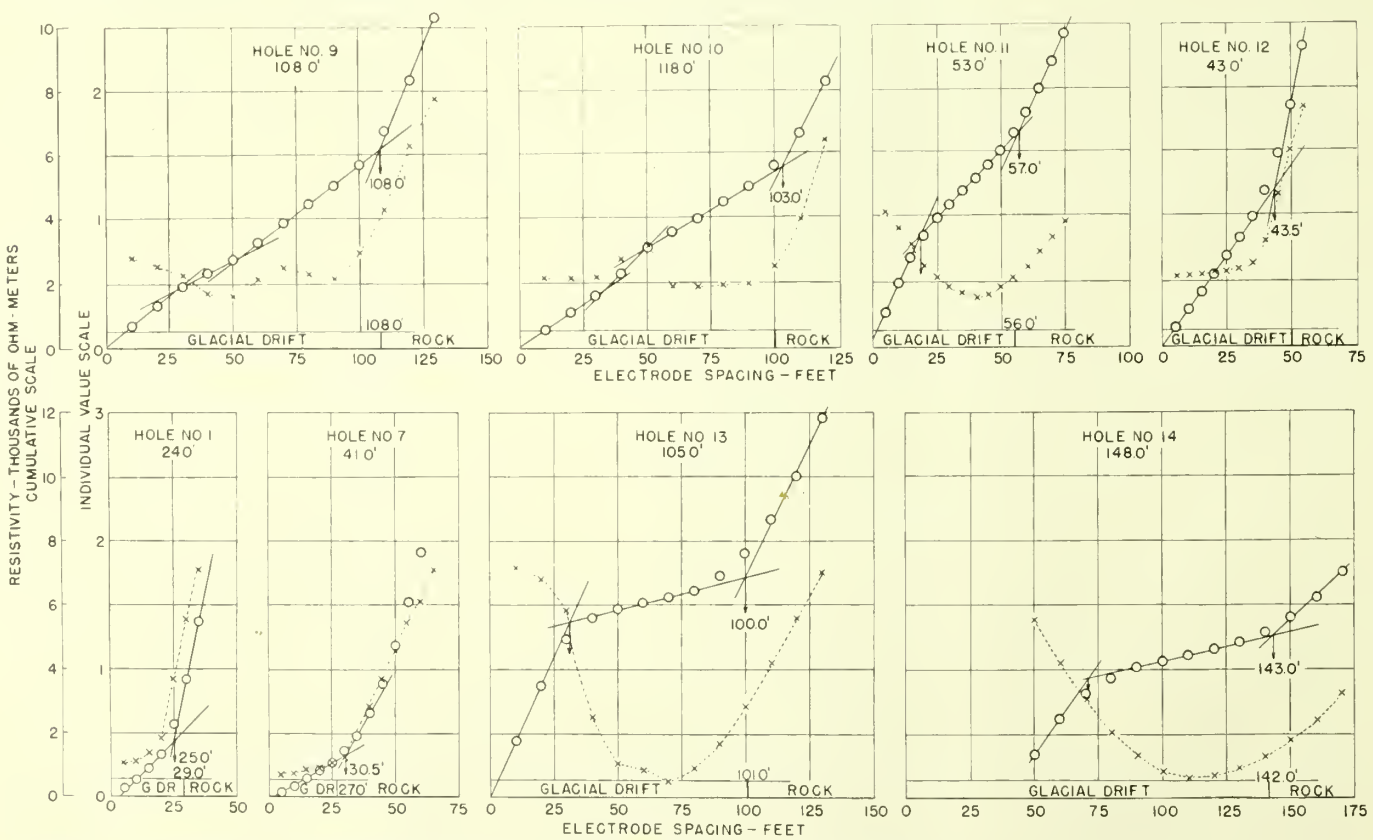


FIGURE 8.—REPRODUCTION OF CURVES PUBLISHED BY I. B. CROSBY AND E. G. LEONARDON. THE DEPTH AS ESTIMATED BY ORIGINAL INVESTIGATORS IS GIVEN AT THE TOP OF EACH DIAGRAM.

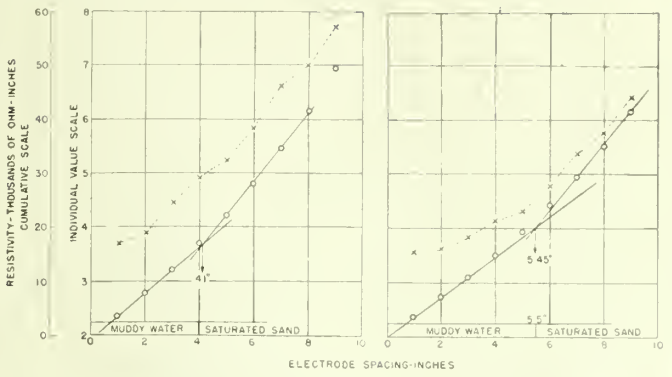


FIGURE 9.—CUMULATIVE RESISTIVITY CURVE METHOD OF ANALYSIS APPLIED TO DATA OBTAINED IN EXPERIMENTS ON MODELS BY T. A. MANHART.

tion 3 apparently cannot be interpreted by any known method.

Figure 7 shows data from a paper by Watson (*8*) in which curves obtained from experiments on models are described. He made experiments in which a carefully prepared sandy shale was screened and mixed with different percentages of moisture to form surface layers having different resistivities. A rubber sheet was placed under the surface layer to simulate a bottom layer having infinite resistivity. Both cumulative curves indicate rather accurately the location of the buried rubber sheet as well as the interface between layers of soil of different resistivity.

Figure 8 contains reproductions of several curves published by Crosby and Leonardon (*1*). The depth to rock as predicted by Crosby and Leonardon is noted near the top of each graph while that estimated by the author is shown on the cumulative curve. The actual depth as obtained by drilling is shown along the lower edge. With three exceptions, the cumulative curve method of analysis checked the actual conditions beneath the surface within about 3 percent. The average error for all curves was 5.8 percent.

No data were included in Crosby and Leonardon's paper relative to variations in the surface layer of material classed as glacial till. It is possible that the presence of a water table is responsible for the breaks in some of the cumulative resistivity curves at depths less than those given for the rock surface.

Curves from a paper by Manhart (*5*) are reproduced in figure 9. In this paper the author reports on experiments in which he made numerous tests on materials placed in a tank under laboratory controlled conditions. The two curves were obtained when using a surface layer of muddy water which was underlain by saturated sand. The smooth curves of apparent resistivity obtained by Manhart lend themselves well to a solution by the method of analysis proposed in this paper. Of the many curves presented by Manhart 13 were analyzed by the proposed method. Four curves represented 2 layers of material and 9 represented 3 layers. A total of 22 depth determinations were made. In only two or three cases were the conditions indicated by the analysis materially different from those established in the laboratory.

In figure 10 a curve given by Roman (*6*) has been replotted as a normal apparent resistivity-electrode spacing curve and also in the form of a cumulative curve. ρ_a, the apparent resistivity, is assumed to be given in ohm-centimeters. Roman suggests that

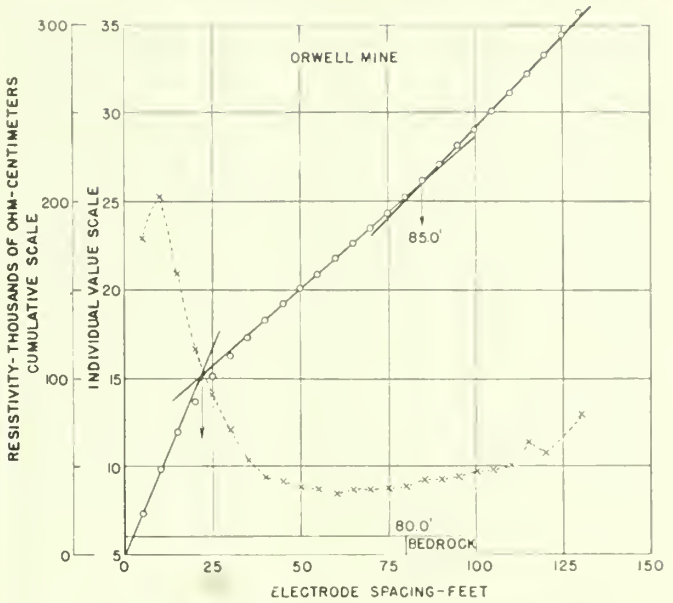


FIGURE 10.—CUMULATIVE RESISTIVITY CURVE METHOD OF ANALYSIS APPLIED TO DATA REPORTED BY ROMAN.

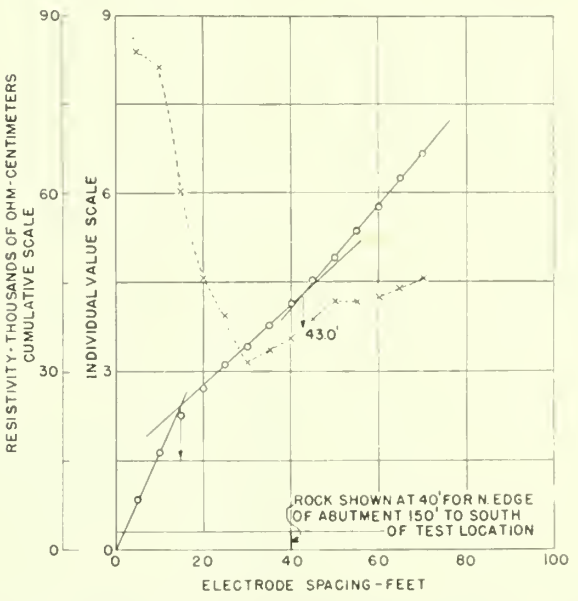

RESISTIVITY-THOUSANDS OF OHM-CENTIMETERS CUMULATIVE SCALE
INDIVIDUAL VALUE SCALE
43.0'
ROCK SHOWN AT 40' FOR N. EDGE OF ABUTMENT 150' TO SOUTH OF TEST LOCATION
ELECTRODE SPACING-FEET

FIGURE 11.—RESULTS OF A RESISTIVITY TEST NEAR THE WEST ABUTMENT OF ARLINGTON MEMORIAL BRIDGE.

further theoretical studies are required to make possible the interpretation of such curves. Although use of the original apparent resistivity and electrode spacing data would have resulted in a more accurate curve for use in the empirical analysis which is proposed, the data shown in figure 10 indicate that the cumulative curve method of analysis has given a reasonably accurate depth determination. The depth of 80 feet to bedrock was given in the discussion which followed the presentation of Roman's paper. Mention also was made in the discussion of the presence of swampy materials on the surface overlaying the hardpan which overlay the rock at the test location. The change indicated in the cumulative curve at a depth of approximately 22 feet may be at the boundary between these two surface materials.

Many other published data have been replotted and analyzed using the cumulative curve method with about

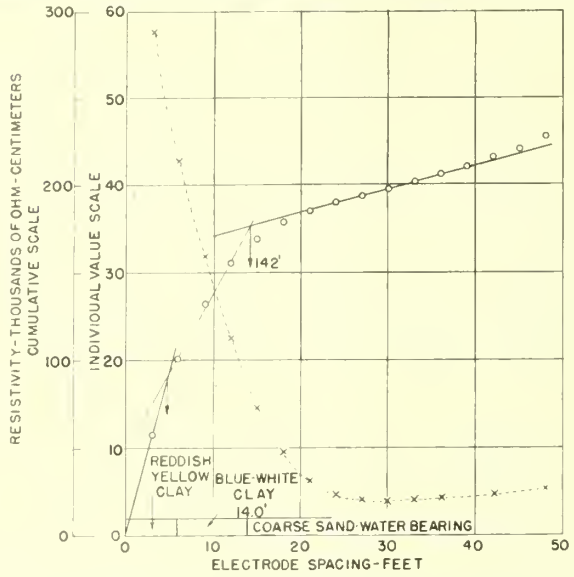


FIGURE 12.—RESULTS OF A RESISTIVITY TEST AT LOCATION OF PROPOSED UNDERPASS NEAR PETERSBURG, VA.

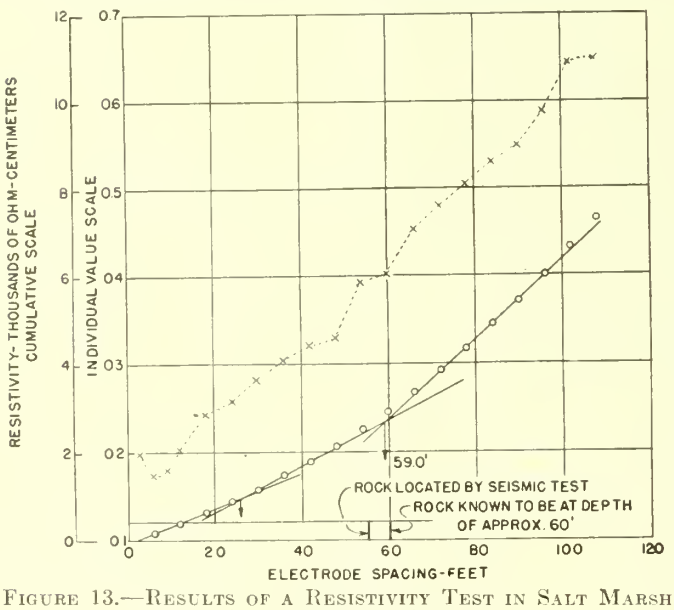


FIGURE 13.—RESULTS OF A RESISTIVITY TEST IN SALT MARSH NEAR HAMPTON, N. H.

the same general degree of success as indicated in the examples given. There are border-line conditions of test which do not lend themselves to an analysis by this method. As stated previously, both the Gish-Rooney and the cumulative curves are used in interpreting the data from a particular test. The general shape of the Gish-Rooney curve serves to indicate the probable ground conditions, whether two-layer or more, and it may also give some clue to the approximate depth of the surface layers.

FIELD TESTS CONDUCTED BY THE PUBLIC ROADS ADMINISTRATION

The results of tests conducted by the Public Roads Administration that involve a variety of surface and subsurface conditions in the vicinity of Washington, D. C., and in New Hampshire are shown in figures 11 to 14. The curve shown in figure 11 was obtained near the west abutment of the Arlington Memorial Bridge across the Potomac River entering Washington, D. C. At this location a few feet of earth fill

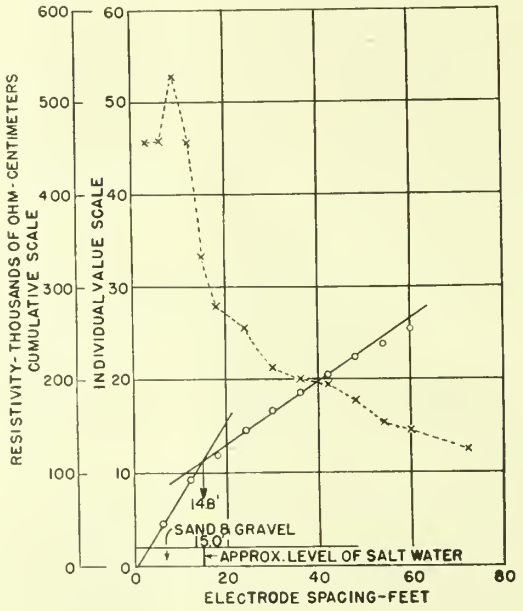


FIGURE 14.—RESULTS OF A RESISTIVITY TEST ON SAND AND GRAVEL FILL APPROXIMATELY 15 FEET ABOVE SEA LEVEL NEAR HAMPTON, N. H.

overlay soft mud which was underlain in turn by a rock formation. The depth of 43 feet to rock indicated by the cumulative curve is in reasonable agreement with the depth of 40 feet found by drilling at the abutment location about 150 feet to the south. The change shown by the cumulative curve at a depth of about 15 feet is probably where the fill material merges with the underlying mud.

Figure 12 shows data from a test where heavy clay was underlain by coarse, water-bearing sand at a depth of 14 feet. The relatively smooth individual value or Gish-Rooney curve shows no indication of the sand stratum at that depth but the stratum is indicated at 14.2 feet on the cumulative resistivity curve.

Figure 13 shows the resistivity curve for a location in the salt marshes near Hampton, N. H., where rock was known to be at a depth of approximately 60 feet. The Gish-Rooney curve shows an increased upward trend at an electrode spacing of 48 feet. The determination with the cumulative curve checked the known depth and the depth indicated by refraction seismic tests made nearby within 1.7 and 6.9 percent, respectively. In the absence of previous knowledge of the general nature of the subsurface formation it may be necessary to put down one or two drill holes to establish the significance of data which show several changes in direction or breaks in the cumulative curve.

Figure 14 shows the result of a test made on a sand and gravel fill near the sea and about 15 feet above sea level at Hampton, N. H. The cumulative curve indicates a low resistivity medium such as salt water at about 15 feet.

Curves representative of tests made with the resistivity apparatus over peat bogs are shown in figure 15. In this series of tests the cumulative curve proved to be a satisfactory means of analyzing the data obtained. These tests are of interest because it seemed probable that the peat and marl formations overlying the sand or hardpan usually comprising the bottom formations of the bogs would approach the uniform

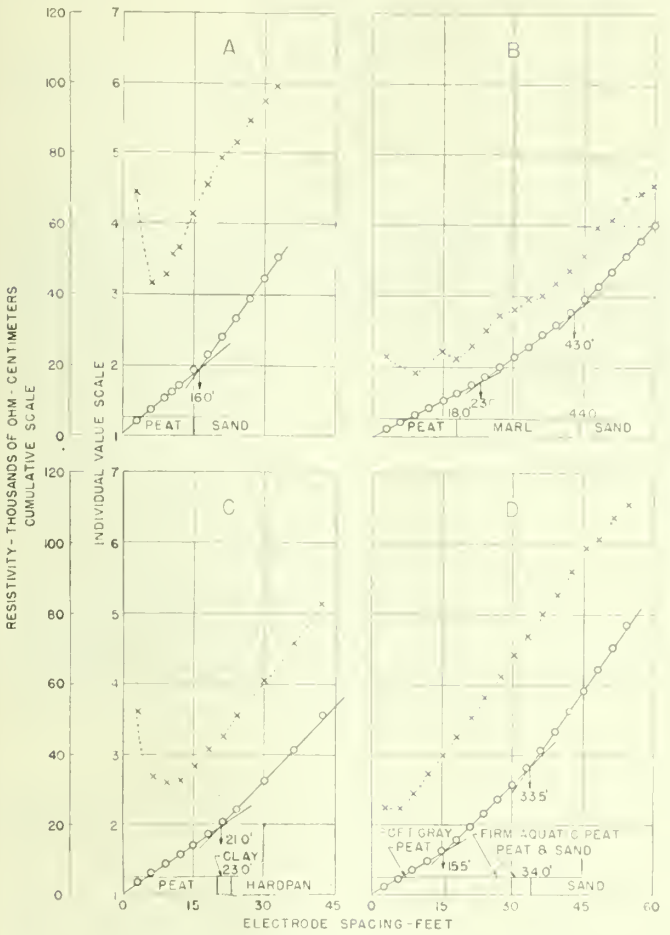


FIGURE 15.—RESULTS OF RESISTIVITY TESTS OVER PEAT BOG FORMATIONS.

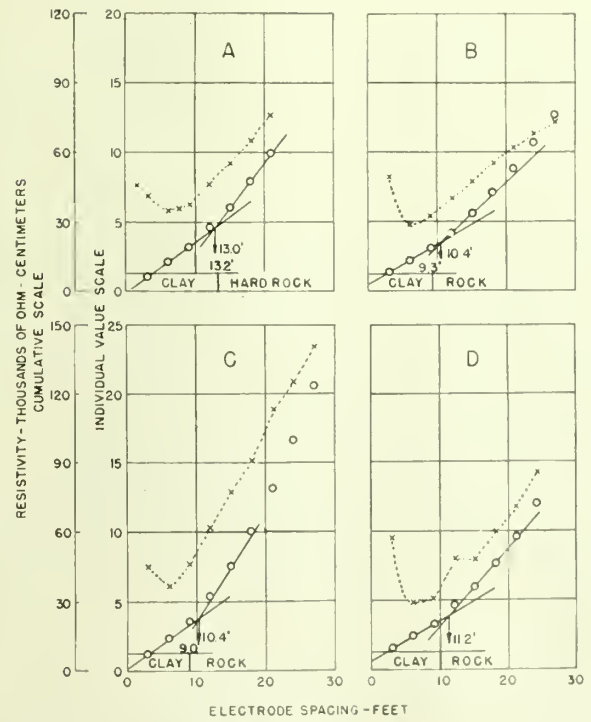


FIGURE 16.—EMPIRICAL METHOD OF ANALYSIS APPLIED TO RESISTIVITY CURVES FOR A CLAY STRATUM UNDERLAIN BY ROCK IN THE VICINITY OF WASHINGTON, D. C.

FIGURE 17.—VIEW OF EXCAVATED AREA SHOWING IRREGULAR SURFACE OF ROCK FORMATION INVOLVED IN RESISTIVITY TESTS.

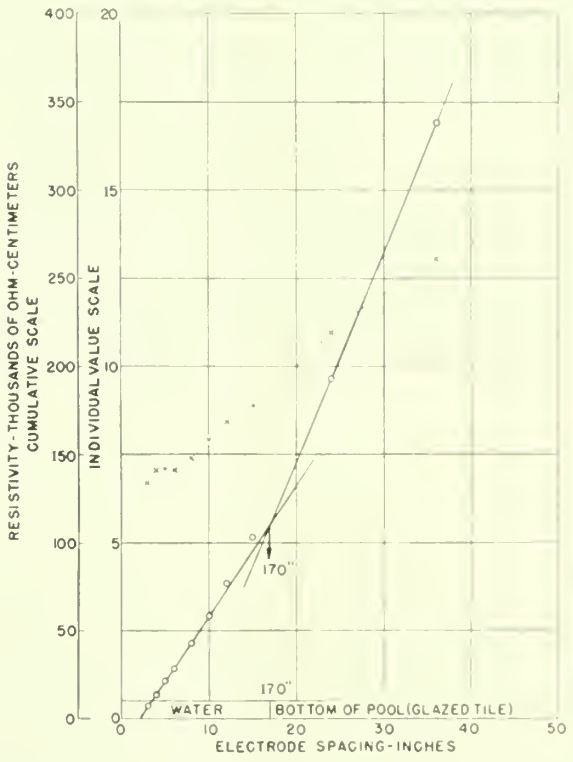


FIGURE 18.—RESULTS OF A RESISTIVITY TEST IN A TILED POOL.

layers of homogeneous materials assumed in theory, thus presenting a condition that would be particularly suitable for a satisfactory analysis by the resistivity method. Furthermore it was relatively simple to check directly the indications of the resistivity tests by means of rod soundings with equipment capable of taking samples at any desired depth.

Figure 16 shows data from an extensive resistivity survey completed in the spring of 1942 in the vicinity of Washington, D. C. Based upon some 475 to 500 depth tests and about 10.5 miles of "resistivity traverse" covering an area of about 150 acres, a map was prepared showing subsurface rock elevations by means of rock surface contours. Numerous check borings, with post-hole augers, and direct observations in exca-

vations made subsequent to the preparation of the map, indicated an average accuracy within 1 to 2 feet for the rock contours as established. The depth of the surface clay varied from about 4 feet to 30 feet. The underlying rock was a granite formation of quite irregular contour, as shown in figure 17. The excavation showed differences in elevation of as much as 6 feet in horizontal distances as little as 6 feet at places where irregularities were more pronounced. Under such conditions the resistivity test should offer more dependable over-all depth values than a limited number of individual borings.

The results of a test conducted in the tiled pool in the plaza of the National Academy of Science, Washington, D. C., are given in figure 18. The Gish-Rooney method of plotting the data showed no indication of the depth of the water in the pool, whereas an analysis with the cumulative curve shows a remarkable correlation with the measured depth of 17 inches.

SUMMARY

The cumulative resistivity curve method of analysis offers a simple and rapid means of determining the depth to an underlying formation where no definite indication is given by the apparent resistivity-electrode spacing curve.

Although best suited to shallow two-layer formations the cumulative curve has been applied with good results to data from relatively deep tests and from both two-layer and three-layer formations. It has proved to be particularly useful in making rapid reconnaissance surveys over relatively large areas and where more detailed and time-consuming methods of analysis are not justified economically. This method of analysis is intended to augment and supplement other methods in current use rather than to displace them.

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(Continued from p. 74)

much superior to other methods that the other methods were largely abandoned. The units burned from 50 to 100 gallons of fuel oil per day depending upon the number of hours that steam was generated. Housing on the trucks prevented freezing on the road and the trucks were ordinarily driven into heated garages when not being operated. Two men were required for operation of each unit.

A disadvantage of the equipment is that it is easily damaged and repairs may be difficult. Delays in making repairs may permit ice formation on the road. Careful and skilled operators are required.

CONCLUSIONS

Lasting improvements for prevention of icing should be adopted in preference to winter control expedients wherever the cost and difficulties of control in winter are large. Such work can be performed best during the summer and early fall. Some of the measures which may be applicable under different conditions are:

(1) Improvement of drainage channels and structures.

(2) Raising the grade of the road.

(3) Construction of dikes to confine flow of water and ice within certain channels or to divert it elsewhere, and also to act as storage dams for ice formation.

(4) Construction of icing basins. Generally, basins are suitable only for very small flows.

(5) Construction of subsurface drains. This is a more expensive but permanent cure. Generally it is not suitable in areas of permafrost.

(6) Construction of induced-icing areas.

It is impracticable to prevent icing entirely and if the roadway surface is to be kept ice free some icing control is necessary. Three general control methods were in use on the Alaska Highway during the 1943-44 winter:

Prevention of icing by artificially warming the flow of water.

Periodic removal of the ice formed.

Periodic construction or opening up of drainage channels to carry the flow which would otherwise result in icing. The last method appears to be the most efficient but under certain conditions it may be necessary to resort to other methods.

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STRUCTURAL EFFICIENCY OF TRANSVERSE WEAKENED-PLANE JOINTS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by EARL C. SUTHERLAND, Senior Highway Engineer and HARRY D. CASHELL, Associate Highway Engineer

FIGURE 1.—THE FULL-SIZE PAVEMENT SLABS IN WHICH THE WEAKENED-PLANE JOINTS WERE INCORPORATED.

FOR SOME YEARS it has been recognized that the formation of transverse cracks in concrete pavements can be eliminated only by the introduction of closely spaced transverse joints. Consequently, there has been a gradual trend toward decreased slab lengths and in the past 15 years, toward the use of transverse weakened-plane or dummy joints as a means of dividing the pavement into slabs of short lengths.

Early in 1940 the Public Roads Administration sponsored a comprehensive, cooperative experimental investigation for the primary purpose of studying the effects of varying the spacing of expansion joints in pavements containing closely spaced weakened-plane contraction joints. Incidental with this investigation, some questions arose regarding the ability of joints of the weakened-plane type to transfer loads and thus to control critical stresses caused by loads acting in their immediate vicinity. Previous research on the subject indicated that aggregate interlock as it occurred in such a joint separating two 20-foot slabs could not be depended upon to control load stresses (*5*).[1] For example the efficiency of the joint, as determined by strain measurements, was negligible in the winter when the maximum width of joint opening existed as a result of contraction of the abutting slabs. Furthermore, even in the summer when the width of opening was a minimum, the stress-reducing ability of the joint was questionable, varying from point to point along its edges due to the irregular and sloping manner in which the pavement fractured beneath the parting groove. Similar tests on a weakened-plane joint containing ¾-inch dowels spaced at 18-inch intervals showed a rather high average efficiency for all seasons of the year.

Since the tests just referred to were of limited scope and left unanswered certain questions, it was decided to include, as part of the cooperative investigation, a study of the structural efficiency of weakened-plane joints. Specifically, the research was planned to determine the structural behavior of joints of this type as affected by: (1) Type of coarse aggregate; (2) size of coarse aggregate; (3) presence or absence of dowel bars; (4) method of producing fracture at the joint; (5) compressive forces acting to close the joint; and (6) width of joint opening.

Earlier research on joints indicates that deflection relations are not a measure of the stress conditions that accompany them (*5*). Therefore, measurements of strain in the concrete are necessary to evaluate joints on the basis of their ability to reduce critical stresses caused by applied loads. Since strain measurements are time consuming and because the pavement at the test site should be protected from changes in temperature and moisture, this part of the investigation was conducted on test slabs that were not subjected to traffic. However, data obtained from the study should furnish exact information relative to the initial efficiency of similar weakened-plane joints incorporated in pavements in actual service. The influence of traffic and other factors acting over a period of time to change the initial effectiveness of joints of this type cannot, of course, be ascertained from these tests.

THE TEST SECTIONS DESCRIBED

The special tests were made by the Public Roads Administration on the concrete test pavement shown in figure 1. The pavement was divided into six sections, each being 30 feet in length, 20 feet in width, and of 8-inch uniform thickness. Each section was definitely separated from those adjoining it, and was divided longitudinally by a deformed metal center joint having ⅝-inch diameter tie bars at 60-inch intervals and divided transversely by a weakened-plane joint of specific design. Thus, each section was subdivided into four panels, each 10 by 15 feet.

[1] Italic numbers in parentheses refer to the bibliography, p. 97.

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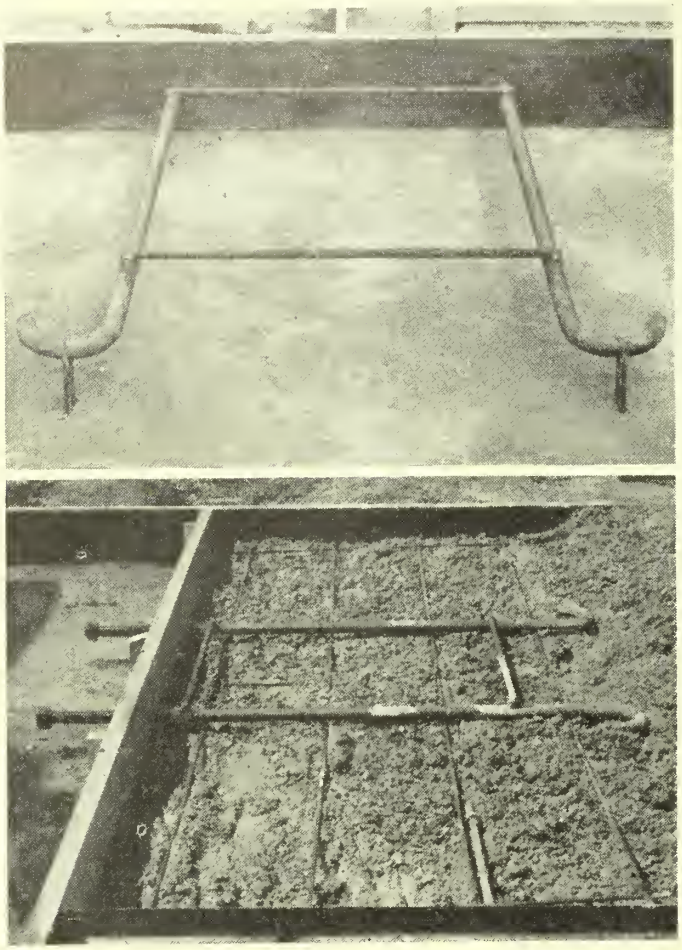

FIGURE 2.—ANCHOR-BOLT ASSEMBLY FOR ATTACHMENT OF APPARATUS USED IN CONTROLLING THE WIDTH OF THE JOINT OPENING.

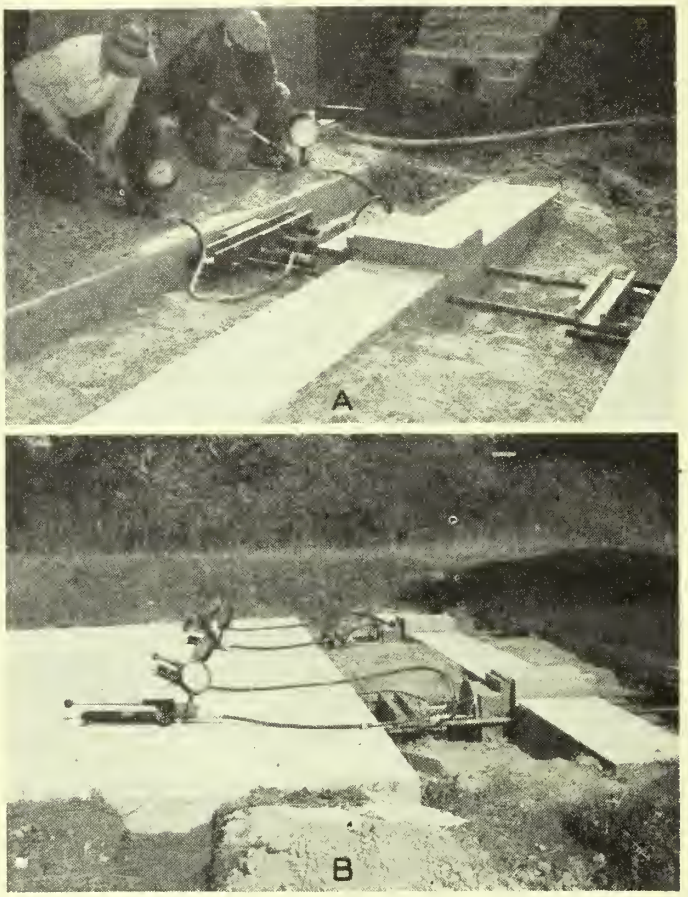

FIGURE 3.—APPARATUS USED FOR APPLYING COMPRESSION AND CONTROLLING THE WIDTH OF THE JOINT OPENING. THE SECTION UNDER TEST IS COVERED BY STRAW IN EACH CASE, (*A*) CLOSING A JOINT AND (*B*) OPENING A JOINT.

At one end of every section, anchor-bolt assemblies, such as shown in figure 2, were embedded symmetrically in the concrete, one on each side of the longitudinal joint, to provide a means for attaching the apparatus used to control the opening of the weakened-plane joints. Between sections 1 and 2, 3 and 4, and 5 and 6 a concrete block or deadman was constructed to provide the reaction whenever direct compression was applied to the sections or whenever the joints were opened.

Figure 3 shows the reaction block together with the apparatus comprising the push-pull system. When closing a joint the jacks exerted pressure directly on the end of the section containing the joint. When opening a joint the jacks were moved to the other side of the reaction block and exerted tension in rods attached to the anchor-bolt assemblies of the section under test. The four hydraulic jacks used in the system were equipped with pressure gages in order to measure the forces applied to the section and, also, to insure uniform forces on both sides of the longitudinal joint.

The Subgrade.—The test sections were laid in the late summer of 1940 on the site of an earlier experimental concrete pavement (*4*). The soil underlying the earlier pavement was classified as a uniform brown silt loam (class A–4). For these later tests, the character of the subgrade immediately beneath the new sections was altered by thoroughly mixing 4 inches of clean sand with the top 4 inches of the original soil. After blending, the material was then compacted at optimum moisture to an average dry density of 136 pounds per cubic foot. This modification of the subgrade soil had the effect of increasing the value of the modulus of subgrade reaction by approximately 10 percent, as determined from load-deflection tests at the interior of the pavement.

The Concrete.—To investigate the influence of type of aggregate on the structural efficiency of the joints, two coarse aggregates having widely different characteristics were used. One aggregate was a siliceous gravel obtained from the Potomac River and the other was a crushed limestone obtained from near Frederick, Md. The gravel was typical of river gravels in that the edges of the individual pieces were well rounded and the surfaces were very smooth. The stone fragments had rather sharp angular fractures and relatively rough surfaces. The difference in character of the two aggregates is shown in figure 4.

The effect of aggregate size was studied for both types of coarse aggregate. This was accomplished by limiting the maximum size of coarse aggregate to 1 inch in some sections and to 2½ inches in others.

The program required the designing of four concrete mixes. For each, the cement was from the same source, the cement factor was approximately six sacks per cubic yard, and the fine aggregate was a Potomac River sand having a fineness modulus of 2.7. Table 1 shows the mix proportions, strength, and other properties of the concrete. The average slump in 17 tests was 2½ inches. Flexure tests were made on 8- by

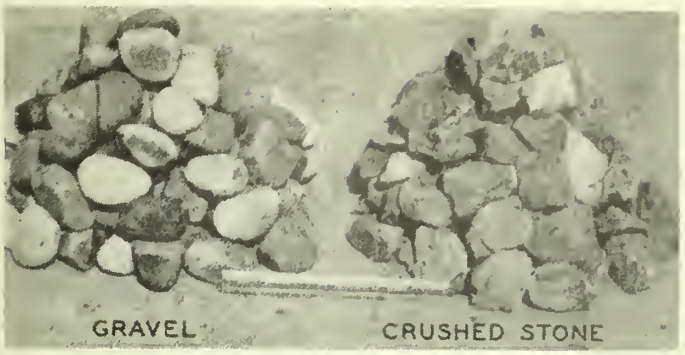

FIGURE 4.—LARGER SIZE OF COARSE AGGREGATE, 2½-INCH MAXIMUM SIZE.

TABLE 1.—*Mix characteristics and strength properties of the concrete*

Type of concrete	Coarse aggregate: Type	Coarse aggregate: Maximum size, (square opening)	Proportions by dry weight	Modulus of rupture: 28 days	Modulus of rupture: 17 months	Crushing strength at 28 days	Modulus of elasticity at 16 months
		Inches	*Pounds*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
I	Gravel	1	94 : 200 : 330	557	627	4,620	4,868,000
II	do	2½	94 : 190 : 340	517	632	4,850	4,650,000
III	Stone	1	94 : 232 : 308	611	725	4,640	5,407,000
IV	do	2½	94 : 222 : 318	530	673	4,520	5,725,000

TABLE 2.—*Features of the weakened-plane joints*

Joint No.	Type of joint (see fig. 5)	Coarse aggregate in adjacent slabs
1	C	Large gravel.
2	A	Small gravel.
3	A	Large gravel.
4	B	Small gravel.
5	A	Small crushed stone.
6	A	Large crushed stone.

8- by 40-inch beams, the load being applied at the third points of a 24-inch span. Compression tests were made on 6- by 12-inch cylinders. All specimens were cured in the same manner as the test sections (as nearly as possible) and, until a few days before testing, were stored in the ground adjacent to the pavement. The modulus of elasticity values were determined from 6- by 12-inch cylinders which had been stored in the ground for a period of 16 months.

THREE TYPES OF WEAKENED-PLANE JOINTS INVESTIGATED

Details of the three types of weakened-plane joints incorporated in the six sections are shown in figure 5. Types A and B are conventional joints. Type C is of special design having, in addition to a shallow surface groove, a bottom metal parting strip which is supposed to fracture the concrete in such a manner that load transfer will be developed to a more positive degree than is possible with the usual type of weakened-plane joint.[2] Figure 6 shows the bottom parting strip in place on the subgrade ready for the concrete to be cast.

Table 2 shows the features of each of the six weakened-plane joints. An examination of the table indicates certain comparisons in which only one variable is present. These are as follows:

[2] This joint was developed by Messrs. L. I. Hewes and William Bertwell.

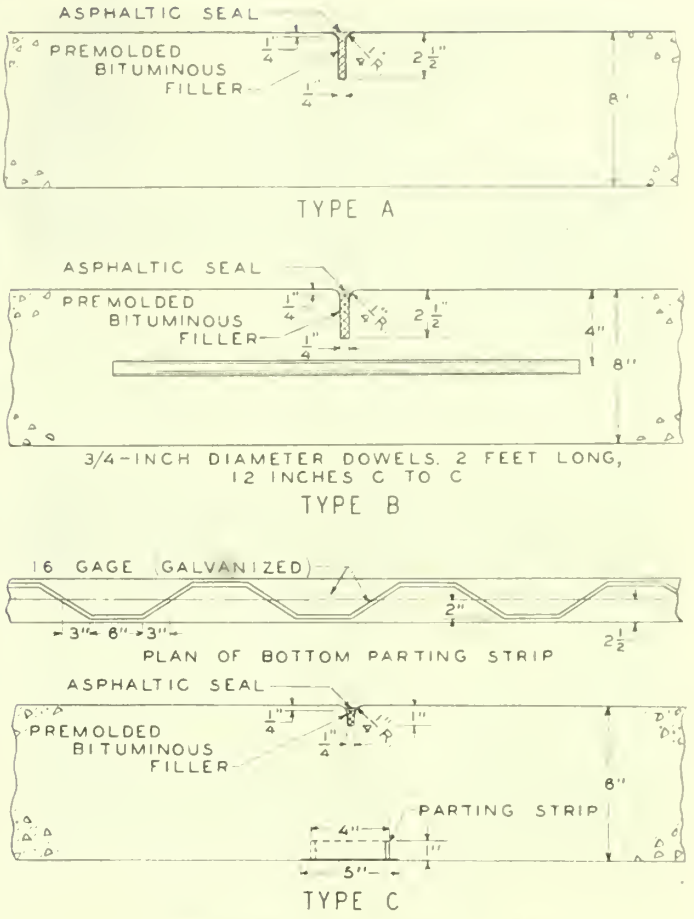

FIGURE 5.—DESIGNS OF WEAKENED-PLANE JOINTS INVESTIGATED.

Type of coarse aggregate:
- A. Small gravel versus small crushed stone (joint No. 2 versus joint No. 5).
- B. Large gravel versus large crushed stone (joint No. 3 versus joint No. 6).

Size of coarse aggregate:
- A. Small gravel versus large gravel (joint No. 2 versus joint No. 3).
- B. Small crushed stone versus large crushed stone (joint No. 5 versus joint No. 6).

Presence or absence of dowel bars:
- A. Joint No. 4 versus joint No. 2.

Method of fracturing pavement:
- A. Joint No. 1 versus joint No. 3.

LOADING AND TESTING PROGRAM SIMILAR TO THAT IN PREVIOUS STUDIES

The technique of testing to determine the efficiency of the various weakened-plane joints was similar to that used in previous studies. Simply, a load of given magnitude was applied successively at selected points at the interiors and free edges of the panels of a section containing a specific joint and at points along the edges of the joint. While this load was acting, the critical strain was measured in the upper surface of the slab. Critical strains occur directly under the contact area in the case of interior loading and, also, when a load is applied at the edges of a joint and at the free edges of a panel but away from the corners. For a load acting at or near the outside joint corners and the free corners of a panel, critical strains occur along the bisector of the corner angle at some distance from the load.

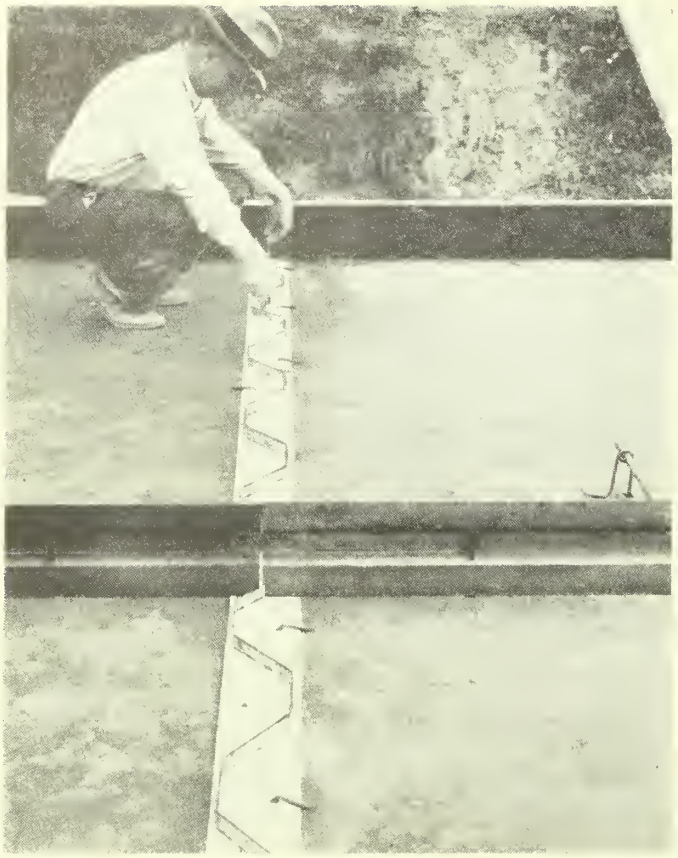

FIGURE 6.—BASE PLATE AND BOTTOM PARTING STRIP OF TYPE C JOINT IN PLACE ON THE SUBGRADE.

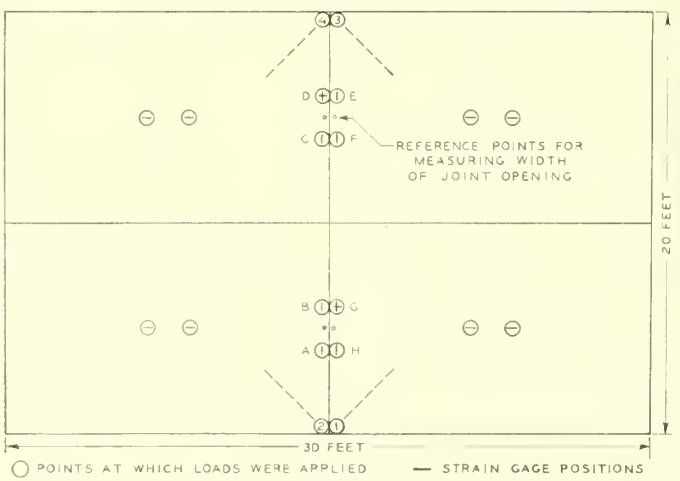

FIGURE 7.—PLAN OF A 20- BY 30-FOOT TEST SECTION SHOWING POINTS AT WHICH LOADS WERE APPLIED AND STRAINS MEASURED.

The points at which the loads were applied and the positions of the strain gages in relation to the load points are shown in figure 7. At each of the indicated points, a load of given magnitude was applied to the surface of the pavement through an 8-inch steel circular bearing plate, the load being maintained for a period of 5 minutes before making strain measurements. The magnitude of the load was measured with a dual-beam dynamometer. A sponge-rubber pad one-half inch in thickness was placed between the rigid plate and a previously prepared smooth surface. This surface was formed at each load point by seating the plate in a thin layer of plaster of paris and then removing the plate after the plaster had set. A large, cylindrical steel tank that transversely spanned the section provided, when partially filled with water, the necessary reaction for the vertical loads. Strains were measured at the positions indicated in the figure with a recording strain gage (*3*) installed between brass posts set into the upper surface of the concrete. The loading equipment and the method of measuring strains are described in detail in an earlier study of the structural behavior of concrete pavement slabs at the Arlington Experiment Farm (*4*).

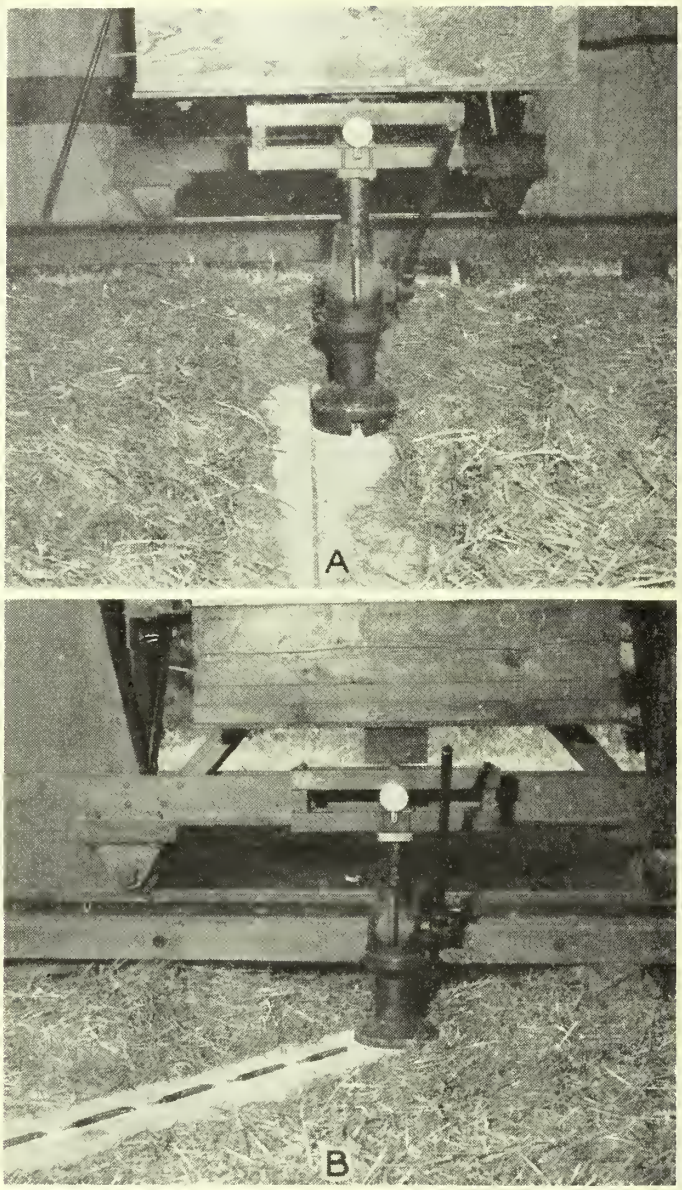

FIGURE 8.—APPARATUS USED FOR APPLYING THE LOAD AND THE ARRANGEMENT OF STRAIN GAGES IN RELATION TO THE APPLIED LOAD: (*A*) LOAD BEING APPLIED AT A JOINT EDGE POINT AND (*B*) LOAD BEING APPLIED AT A JOINT CORNER.

Figure 8 shows the load being applied and the strain gages in place at the joint edge (*A*) and the joint corner (*B*) of one of the test sections. In the case of edge loading, it will be noted that the bearing plate is grooved to accommodate the strain gage. The straw covering shown in the figure extended over the entire section and remained in place during the testing period. In addition, the section was sheltered from rain and direct sunlight. These protective measures minimized

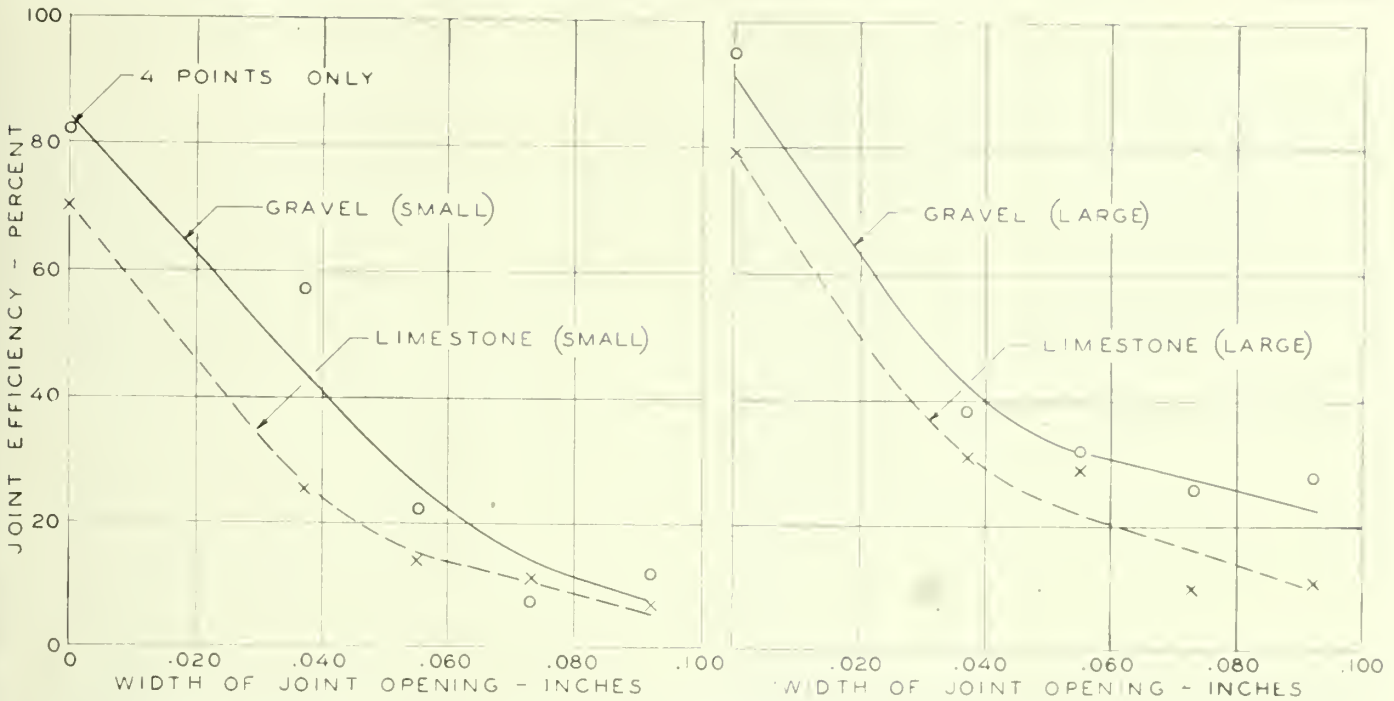

FIGURE 9.—EFFECT OF TYPE OF COARSE AGGREGATE ON THE RELATION BETWEEN JOINT OPENING AND JOINT EFFICIENCY AS DETERMINED WITH EDGE LOADING. EACH VALUE IS AN AVERAGE FROM TESTS AT POINTS A TO H, INCLUSIVE.

moisture changes in the subgrade and temperature warping of the slab.

TESTING SCHEDULE DESCRIBED

Testing of the various joints was conducted between June and October of 1941, 10 to 14 months after the pavement was laid. The testing schedule was essentially the same for each of the joints. Briefly, this schedule consisted of:

1. Measurement of the critical strains at the interiors of the four panels of a given section for a load of selected magnitude.

2. Application of a total compressive force of 120,000 pounds to one end of the section and, with the force acting, measurement of the critical strains at the joint edge points and at the joint corners.

3. Release of compressive force and measurement of the critical strains at the joint for successive controlled widths of opening of 0.037, 0.055, 0.073, 0.092, 0.110, and 0.220 inch. These are computed maximum joint openings which might occur during the annual cycle of temperature and moisture variations for slab lengths ranging from 10 to 60 feet. Initially, it was intended to measure strains both at the edge points and corners of the joints for each of the preceding openings; but, because of a time limit imposed by the necessity for vacating the site, the original schedule was curtailed. The schedule was reduced by obtaining strains at the edges of the joints for openings of 0.037, 0.055, 0.073, and 0.092 inch; and, with the exception of joint No. 2, at the corners for openings of 0.037, 0.073, 0.110, and 0.220 inch.

4. Opening of the joint to a width of one-half inch and measurement of the critical strains at the edge points and corners. For this opening all effects of aggregate interlocking had disappeared and the edges of the joint were acting as free slab ends. In the case of the doweled joint, all dowels were cut before testing at this opening.

5. Remeasurement of the critical strains at the interiors of the panels.

Before presenting the results of this study, attention is called to the fact that the zero widths of the joints are the widths established when the joints were closed by the 120,000-pound compressive force. Prior to the application of this force, it was observed that, with the exception of joint No. 1, the joints were opened about 0.015 inch. Under the compressive force the joints closed approximately 0.010 inch; thus, at the zero width there existed a residual opening of about 0.005 inch. The base readings at the zero joint width and subsequent determinations of the widths of joint opening were obtained with a vernier caliper by measuring between gage plugs cemented into the surface of the slab (see fig. 7).

EFFICIENCY OF THE JOINTS WITH A LOAD ACTING AT THE JOINT EDGE DETERMINED

Previous research has established that, for a slab of uniform thickness, the magnitude of the critical stress produced by a given load is minimum at the interior and maximum at the free edge. Consequently, in this investigation, the ability of the various joints to reduce stresses caused by a load acting at the joint edge, but away from the corners, will be determined by comparing the critical stress at the joint edge with that produced by the same load acting at the free end and at the interior of the slab.

From critical strains measured with the load acting at the aforementioned points, the efficiency of a joint may be computed by the following equation:

$$E = \frac{\sigma_f - \sigma_j}{\sigma_f - \sigma_i}$$

in which E = joint efficiency;
σ_f = critical stress for a given load applied at the free end;

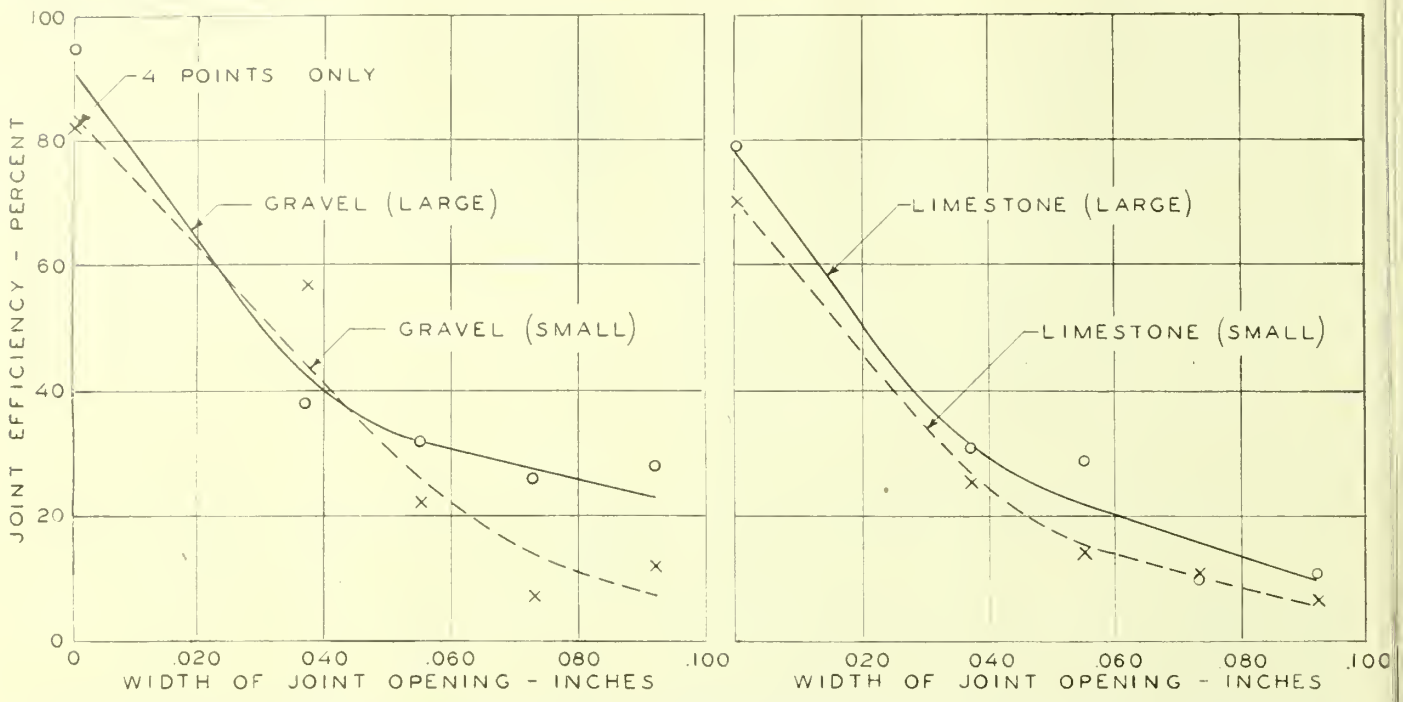

FIGURE 10.—EFFECT OF SIZE OF COARSE AGGREGATE ON THE RELATION BETWEEN JOINT OPENING AND JOINT EFFICIENCY AS DETERMINED WITH EDGE LOADING. EACH VALUE IS AN AVERAGE FROM TESTS AT POINTS A TO H, INCLUSIVE.

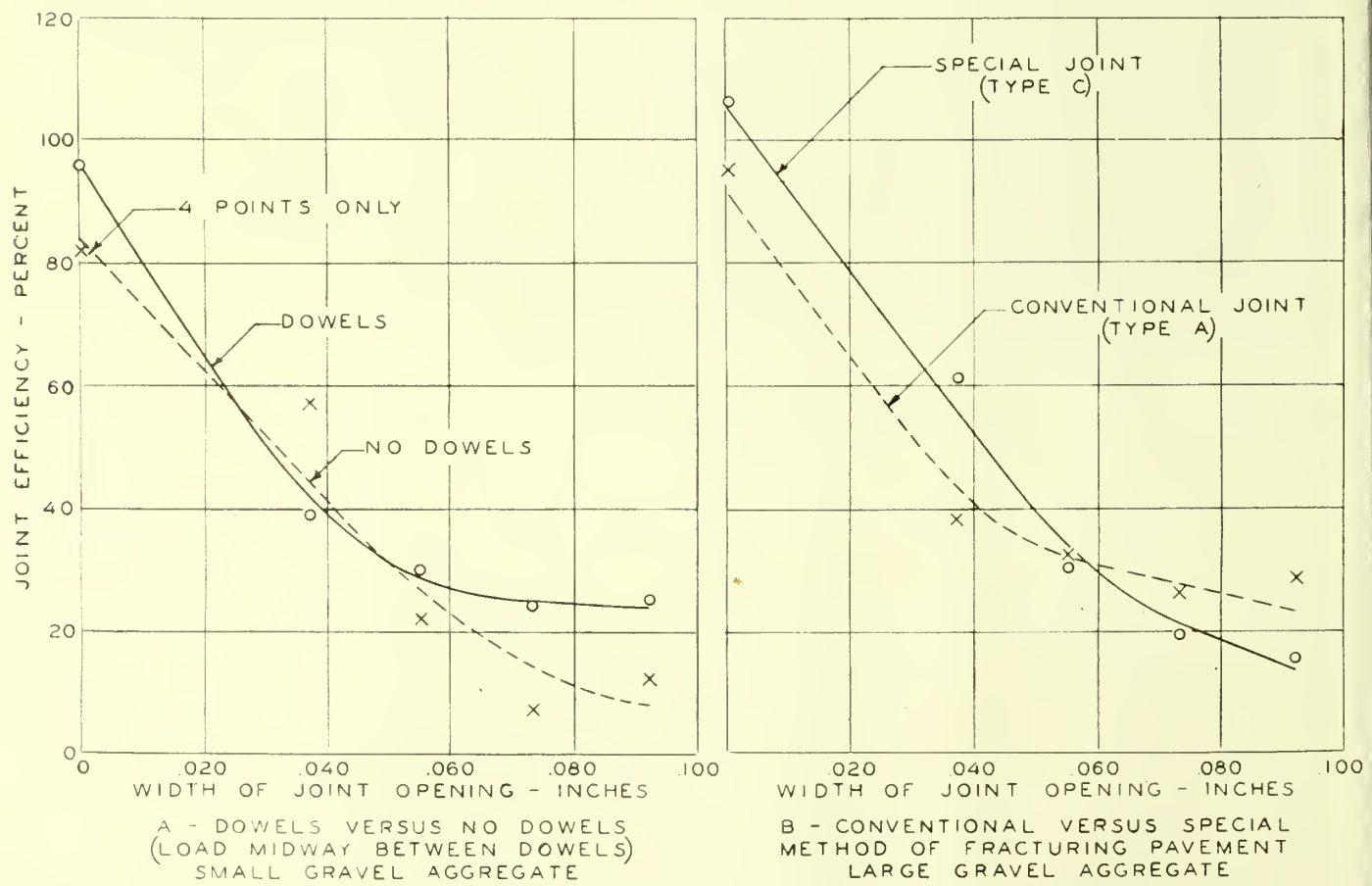

FIGURE 11.—EFFECT OF DESIGN DETAILS ON THE RELATION BETWEEN JOINT OPENING AND JOINT EFFICIENCY AS DETERMINED WITH EDGE LOADING. EACH VALUE IS AN AVERAGE FROM TESTS AT POINTS A TO H, INCLUSIVE.

σ_j= critical stress for a given load applied at the joint edge;
σ_i= critical stress for a given load applied at the interior of the slab.

It will be noted that this equation indicates an efficiency of 100 percent if the critical stresses at the joint edge and at the interior are equal and, conversely, a joint efficiency of zero, if the critical stress at the joint edge is equal to that of the free end.

Figures 9, 10, and 11 show relations between edge efficiency and width of joint opening for each of the six weakened-plane joints listed in table 2. These relations are grouped in accordance with the comparisons given in connection with the discussion of table 2. Referring to the efficiency formula, the value of σ_i for a given section is based on the average of the strains measured (both before and after testing the joint) at the eight interior points of the slab units (see fig. 7). Similarly, the σ_j values and the value of σ_f for each joint are established from the strains measured successively at the eight edge points, *A* to *H*, inclusive, the strains for σ_f being measured after the joint had been pulled apart one-half inch.

The relations just presented show that efficiencies within the approximate range of 70 to 100 percent were attained by the various joints at the zero width of joint opening or, in other words, when the 120,000-pound compressive force was acting. If it is considered that the total force, used to close the joints, was uniformly distributed over the 5½-inch depth of the joint face below the parting grooves of joint types A and B, the average compression would be only 91 pounds per square inch. This value might even be reduced slightly if the subgrade resistance were taken into account. Hence, it appears that all of the weakened-plane joints studied in this investigation attained high efficiency in the presence of interfacial pressure much less than might develop in pavements under conditions of restrained expansion.

As previously mentioned, there was a residual joint opening of about 0.005 inch when the joints were tested while under compression. After completion of the regularly scheduled tests, joints Nos. 3 and 6 were closed and again tested while subjected to the 120,000-pound compressive force. Because of slight changes in the relative elevation of abutting slabs, the rough joint faces did not match as perfectly as before and the residual joint openings generally were much greater than those that existed at the time of the initial tests. At joint No. 3, a separation of 0.14 inch remained after application of the compressive force while at joint No. 6 the separation was 0.02 inch. It is of interest that the efficiencies of joints Nos. 3 and 6, which were initially 95 and 79 percent respectively, were still 61 and 49 percent respectively when retested under the rather severe conditions described above. These data indicate that weakened-plane joints when subjected to direct compressive forces are able to maintain fair efficiencies, even if the fractured faces are not perfectly matched.

Another observation made during the testing of the joints was that, in the presence of the 120,000-pound compressive force, the stress directly under the applied vertical load in a direction perpendicular to the edge of the joint was about one-half of the stress in a direction parallel to the edge of the joint. As the stress for a comparable load acting at a free end is virtually zero

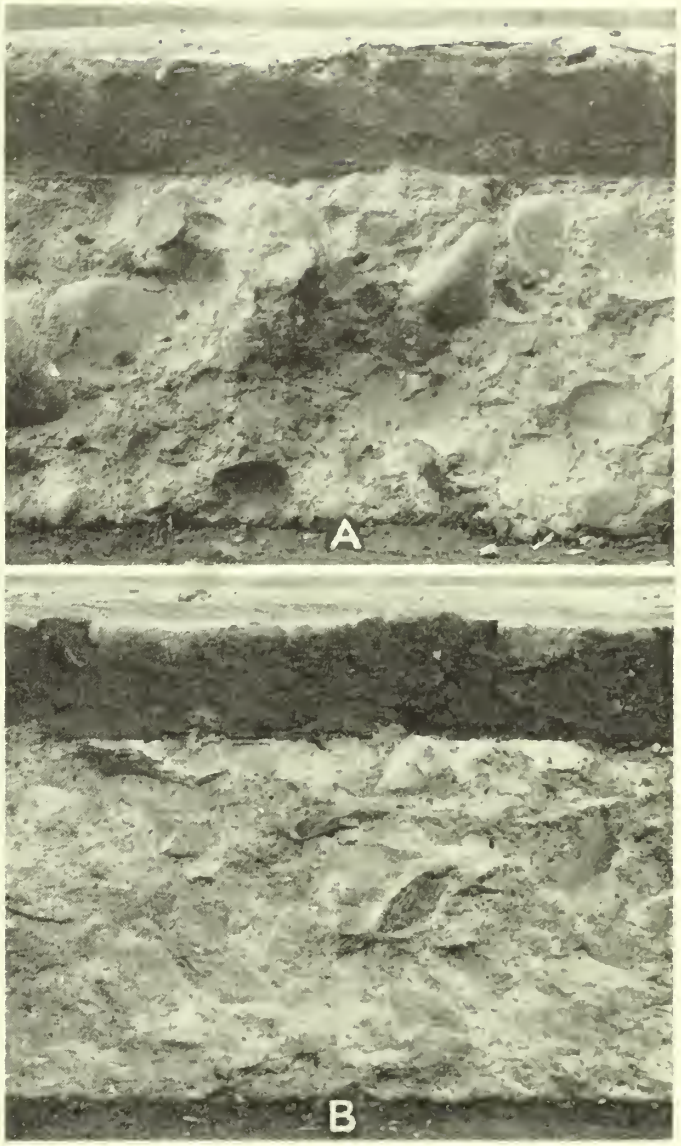

FIGURE 12.—EXPOSED FACES OF WEAKENED-PLANE JOINTS. (*A*) SECTION 3 CONTAINING LARGE GRAVEL COARSE AGGREGATE AND (*B*) SECTION 6 CONTAINING LARGE CRUSHED-STONE COARSE AGGREGATE.

in the perpendicular direction, the tests indicate that the compressive force makes it possible for the joint to transmit appreciable bending moment in addition to shear.

Again referring to the relations of figure 9, 10, and 11, it will be observed that the efficiencies of the joints decreased progressively as the width of joint opening was increased. At an opening of 0.037 inch, the efficiencies for the various joints, as established by the curves, ranged from 27 to 56 percent. At an opening of 0.092 inch, none of the joints had an efficiency greater than 25 percent.

JOINT EFFICIENCY INFLUENCED BY ROUGHNESS OF JOINT FACES

Figure 9 shows that the joints in sections containing the gravel coarse aggregate developed higher efficiencies than similar joints in sections having comparable sizes of the crushed-stone coarse aggregate. At some of the joint openings the difference was as much as 15 percent.

In an attempt to determine the cause of this difference in efficiency, a portion of one of the panels forming

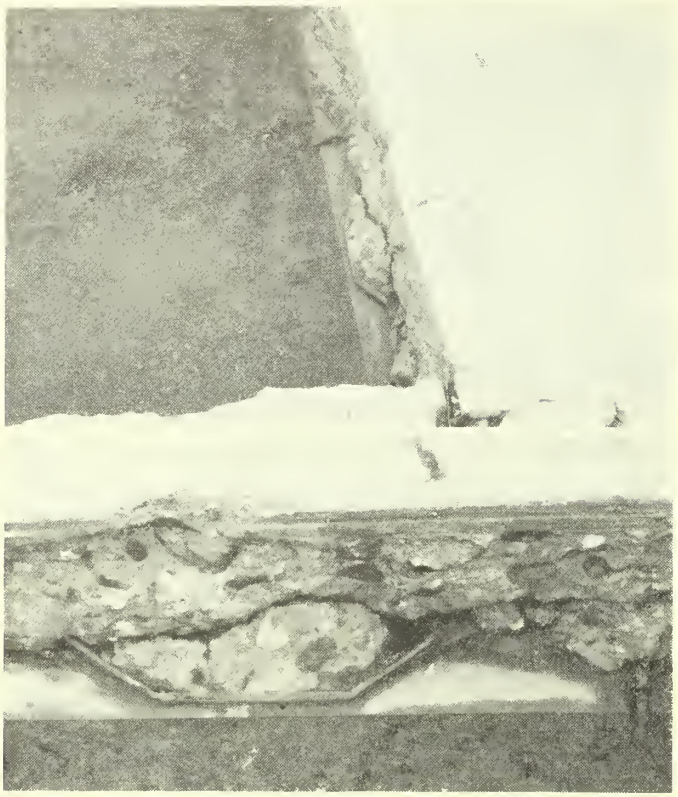

FIGURE 13.—EXPOSED EDGE OF TYPE C WEAKENED-PLANE JOINT.

the joint was removed from each section so that the faces of the joints could be examined. This examination showed that in the sections having gravel coarse aggregate the joint faces were rougher than those of sections containing crushed-stone coarse aggregate due to the fact that when the crack occurred at the joints, the crushed-stone aggregate fractured whereas the gravel aggregate, in most instances, pulled out through loss of bond. This contrast in the surfaces of the fractured faces is shown in figure 12. The rough faces of the joints in sections containing gravel aggregate apparently were better able to transmit shear than were the relatively smooth faces of the joints in sections having crushed-stone aggregate.

In connection with the preceding discussion of the relative roughness of the joint faces, it is of interest to refer to the previously mentioned widths of opening that existed at joints Nos. 3 and 6 after they were closed upon completion of the regularly scheduled tests. It will be recalled that the residual opening of joint No. 3 (large gravel aggregate) was 0.14 inch but that of joint No. 6 (large crushed-stone aggregate) was only 0.02 inch.

Figure 10 offers a comparison between the efficiency relations of the two sizes of gravel aggregate and of the two sizes of crushed-stone aggregate. In the case of the gravel aggregate, the two efficiency curves are practically coincident for a width of joint opening up to 0.05 inch, after which the joint in the section containing the large-size gravel definitely was more efficient. This greater efficiency was probably due to the greater projections of the larger size of gravel aggregate. In the case of the crushed-stone aggregate, the joint in the section having the large-size stone was slightly, but only slightly, more efficient than a similar type of joint in the section built with the small-size stone. The probable reason for this condition is that both the large- and small-size crushed-stone aggregate fractured as the crack formed, thereby producing in both instances equally smooth joint faces.

In order to evaluate the effects of dowels on the relation between joint efficiency and joint opening, comparative efficiency data were established for two joints which were similar in every respect except that one contained dowels. These are shown in figure 11, *A*. Both of the joints were in sections having small gravel aggregate. Figure 5, shows that the dowels were three-fourths inch in diameter, 24 inches long, and were spaced at 12-inch intervals. The load was applied midway between dowels or at weakest locations along the joint edge. Any departure of the efficiency curve of the doweled joint from the curve of the undoweled joint is presumed to be caused by the action of the dowels.

The general coincidence of the two curves at openings between zero and 0.05 inch, inclusive, indicates that in this range the dowels aided but little the natural interlocking of the aggregate in the joint faces. However, after an opening of 0.05 inch the dowels had a marked effect on joint efficiency.

It will be recalled that earlier in this report mention was made of a previous study of a weakened-plane joint containing dowels spaced at 18-inch intervals. This study showed the joint to be 65 to 70 percent efficient for both summer and winter. As shown by the curve of figure 11, *A*, the doweled joint of the present investigation was only 25 percent efficient at openings greater than 0.06 inch. With our present knowledge derived from experimental studies of the structural behavior of dowels in transverse joints, this difference in efficiency cannot be fully explained. When more data are available a definite reason may be forthcoming. For instance, future research may shed some light upon the influence of subgrade stiffness and of pavement thickness on the structural performance of dowels. In the case of the two doweled joints in question, attention is called to the fact that the test pavement of the investigation being reported was not only constructed upon a somewhat more rigid subgrade; but was of 8-inch uniform thickness as compared to an approximate 7-inch thickness that existed at the point of test in the earlier study.

JOINT EFFICIENCY VARIES AT INDIVIDUAL POINTS ALONG JOINT EDGE

The efficiency relations of figure 11, *B* provide some measure of the effectiveness of the type C joint as compared with the conventional type of joint. Both of the joints in this comparison were in sections containing large gravel aggregate. The relations indicate that a clear-cut advantage of one type of joint over the other does not exist. At a joint opening of about 0.06 inch, both are equally efficient. At openings less than 0.06 inch, the special type of joint is more efficient than the conventional type; but at openings greater than 0.06 inch, the reverse is true.

Just prior to the testing of each of the joints, the earth shoulder was removed for the purpose of examining the crack that had formed. As remarked before, the crack that occurred at the special joint, No. 1, was unlike those that developed at the other five joints. Specifically, this crack was extremely fine and was visible for only one-half the depth of the slab; whereas, the cracks at the other joints were very definite, being open about 0.015 inch. Moreover, when the 120,000-

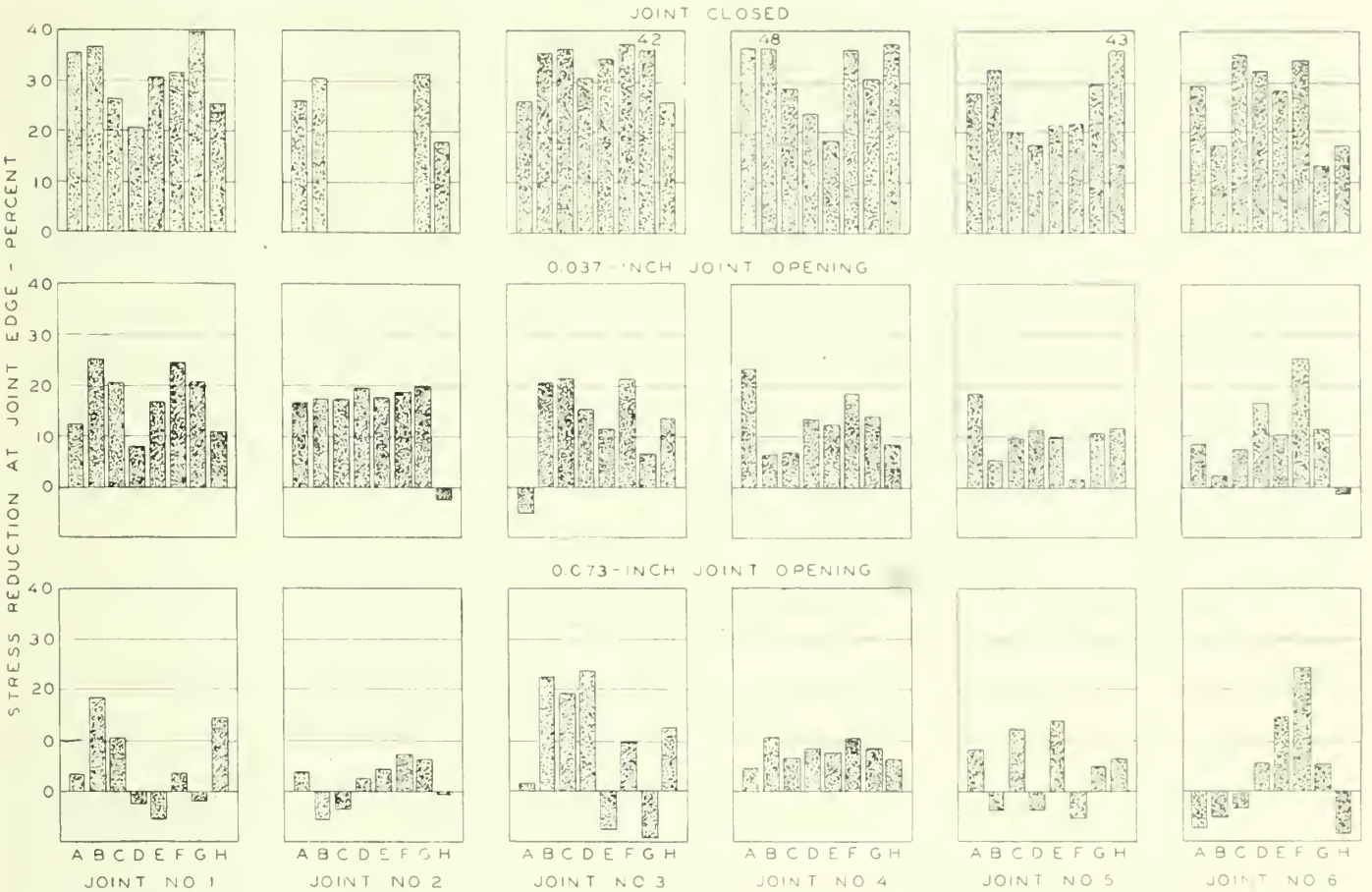

FIGURE 14. VARIATION IN THE AMOUNT OF STRESS REDUCTION AT INDIVIDUAL POINTS A TO H, INCLUSIVE, ALONG SLAB EDGES AT WEAKENED-PLANE JOINTS. CLOSED CONDITION IS THAT CREATED BY A COMPRESSIVE FORCE OF 120,000 POUNDS APPLIED AT ONE END OF THE SECTION.

pound compressive force was applied, the special joint closed only 0.003 inch as compared to a closure of 0.010 inch that was measured at the other joints. Besides the differences just mentioned, the special joint could be pulled apart only 0.006 inch, even by a total force of 110,000 pounds which, incidentally, started the 20- by 30-foot section to slip as a whole over the subgrade. As a consequence, wedges were used in conjunction with the pulling force to effect a complete separation.

The difference in the behavior of the special joint as compared with that of the other joints is an indication that the bottom parting strip failed to fracture the pavement as planned. Figure 13 shows the exposed joint edge of the special joint after a section of the pavement adjacent to the joint had been removed. This illustration shows rather conclusively that the bottom parting strip did not function in the intended manner.

The efficiencies of the various joints have thus far been discussed on the basis of relations determined from the averages of strains obtained at a number of load points. Figure 14 shows the amount of stress reduction found at individual points along the edges of the joints. These reduction values are expressed as percentages and are based on the stress determined at the points when the joint edges were acting as free slab ends. Since strains were measured at the same load point for all widths of joint opening and, also, for the basic free-end condition, the reduction values at the individual points, as shown in figure 14, should be affected but little by variations in the condition of the concrete and in subgrade support.

An estimate of the ability of the joints to reduce the critical stresses at the individual load points is provided by comparing their stress-reduction values with a similar value determined for the interiors of the sections. This interior stress-reduction value was found to be 33 percent, being established from the average of all of the critical strains obtained at the free ends and at the interiors of the panels of the six sections. Thus, if a reduction of 33 percent is attained at any point along the joint edge, then the joint at that point is assumed to be 100 percent efficient.

Referring to figure 14, it is observed that considerable variation in stress reduction was found at the individual points along the edges of the joints. Nevertheless, in spite of these variations, when the joints were closed and the compressive force acting, a relatively high degree of efficiency was developed at all of the points. At a joint opening of only 0.037 inch, however, the individual stress-reduction values indicate that aggregate interlock was not very dependable in reducing critical load stresses. When the joints were opened to a width of 0.073 inch, their stress-reducing ability diminished to such an extent that no reduction in stress was found at one-third of the points. In this respect all of the joints, with the exception of joint No. 4 which contains dowels, show a zero efficiency at two or more of the individual load points. The nonuni-

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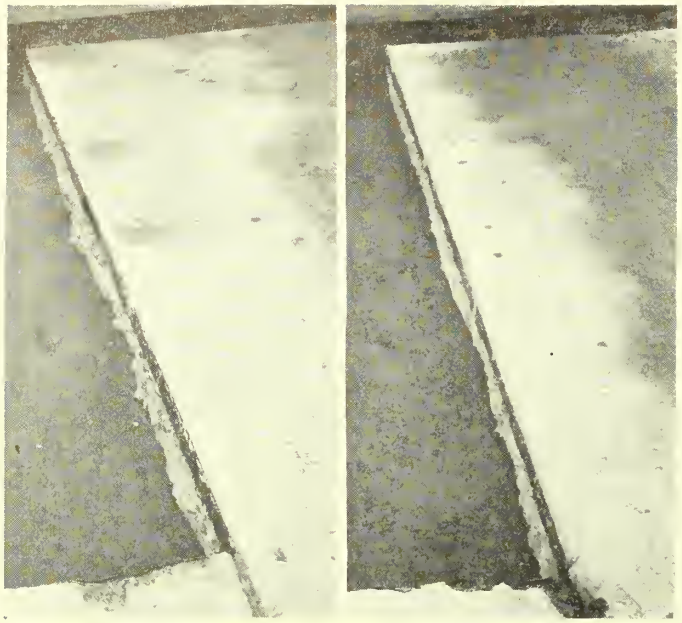

FIGURE 15.—EXPOSED EDGES OF TWO OF THE WEAKENED-PLANE JOINTS, SHOWING IRREGULAR LINE OF FRACTURE.

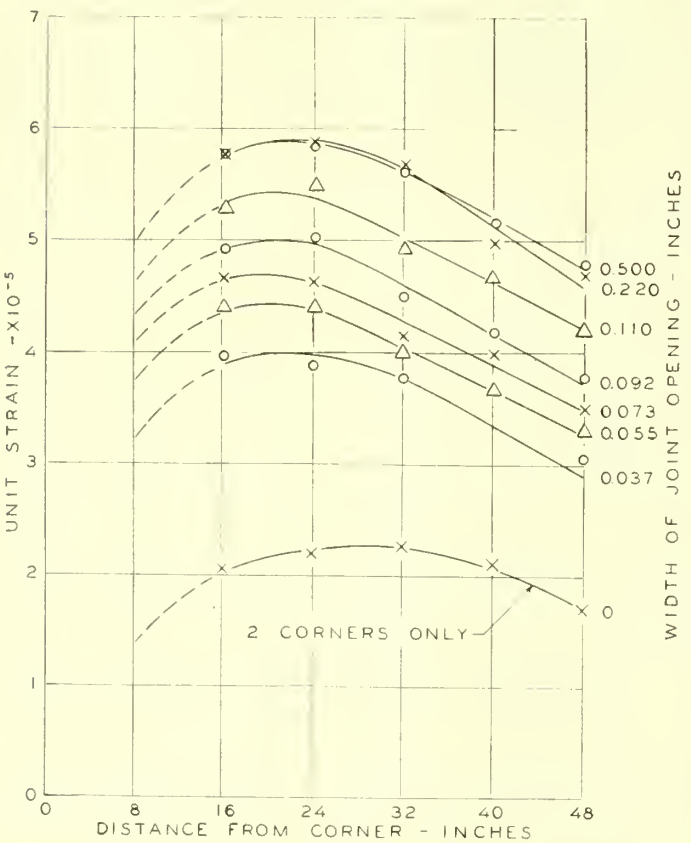

FIGURE 16.—EFFECT OF WIDTH OF JOINT OPENING ON THE MAGNITUDE AND DISTRIBUTION OF STRAIN ALONG THE BISECTOR OF THE CORNER ANGLE AS DETERMINED WITH CORNER LOADING. EACH VALUE IS AN AVERAGE FROM TESTS AT FOUR CORNERS OF JOINT NO 2.

formity of the stress-reduction values is not surprising when one considers the irregular manner in which cracks formed at joints of this type. Figure 15 shows the fractured surfaces at two of the test joints.

Summarizing the preceding data on the effectiveness of the various joints in controlling critical stresses originating from loads acting at some distance from the joint corners, it is concluded that: (1) in the presence of interfacial pressure, all of the weakened-plane joints were effective, even at relatively large residual joint openings; (2) without interfacial pressure, aggregate interlock was found to be an uncertain means of stress control regardless of the type and size of coarse aggregate; (3) the doweled joint showed a rather low average efficiency (about 25 percent) at widths of joint opening greater than 0.06 inch, but the presence of the dowels improved the uniformity of stress reduction at individual points along the edge of the joint; (4) in general, the rougher the joint face the greater the ability to reduce the critical load stress; and (5) roughness of joint faces depends in part upon whether or not the aggregate fractures or pulls out through loss of bond at the time the crack is formed.

EFFECTIVENESS OF THE JOINTS WITH A LOAD ACTING AT THE JOINT CORNER

The ability of the various weakened-plane joints to reduce critical stresses caused by loads acting at their edges has, heretofore, been confined to a discussion of the stresses that occur when the loads are applied at some distance from the joint corners. Under such a loading the critical stress is directly beneath the load and in a direction parallel to the joint edge. When loads are applied at or near the free corners of slabs of constant thickness, critical tensile stresses, as remarked before, develop in the upper surface of the slabs along the bisectors of the corner angles at some distance from the load.

As mentioned earlier, strains were measured along the bisectors of the outside corners of the six weakened-plane joints. These strains afford a means of determining how effectively the joints functioned in reducing the critical stresses for the condition of corner loading. Typical data obtained from tests at one of the joints are shown in figure 16. It is indicated that, for a given width of joint opening, the strains remain nearly constant over the distance between 16 and 32 inches from the corner, the maximum variation within these limits being less than 10 percent.

The data for zero width of joint opening were obtained with the joint under compression which explains why values for this condition are so much smaller than those found with the joint open a small amount.

The effect of type and size of coarse aggregate and of other variables on the reduction in the corner stresses at the various joint openings is given in figures 17, 18, and 19. Comparisons between the different joints in these three figures are the same as those previously described for the corresponding figures in the discussion of the edge condition of loading. The stress-reduction values shown in the graphs are expressed as percentages and were obtained by the following method: The average strains measured at the four corners of a given joint were first plotted in the manner shown in figure 16. From these curves the maximum average strain value for each joint opening was obtained. Using the maximum average strain value for a joint opening of 0.500 inch as a base, the amount of stress reduction effected by the joint at each of the lesser joint openings was then computed for each type of joint.

It is observed that the stress reduction at zero opening, or with the joints under compression, generally exceeded 50 percent of the stress for the free-corner condition. Theoretically, the maximum amount of

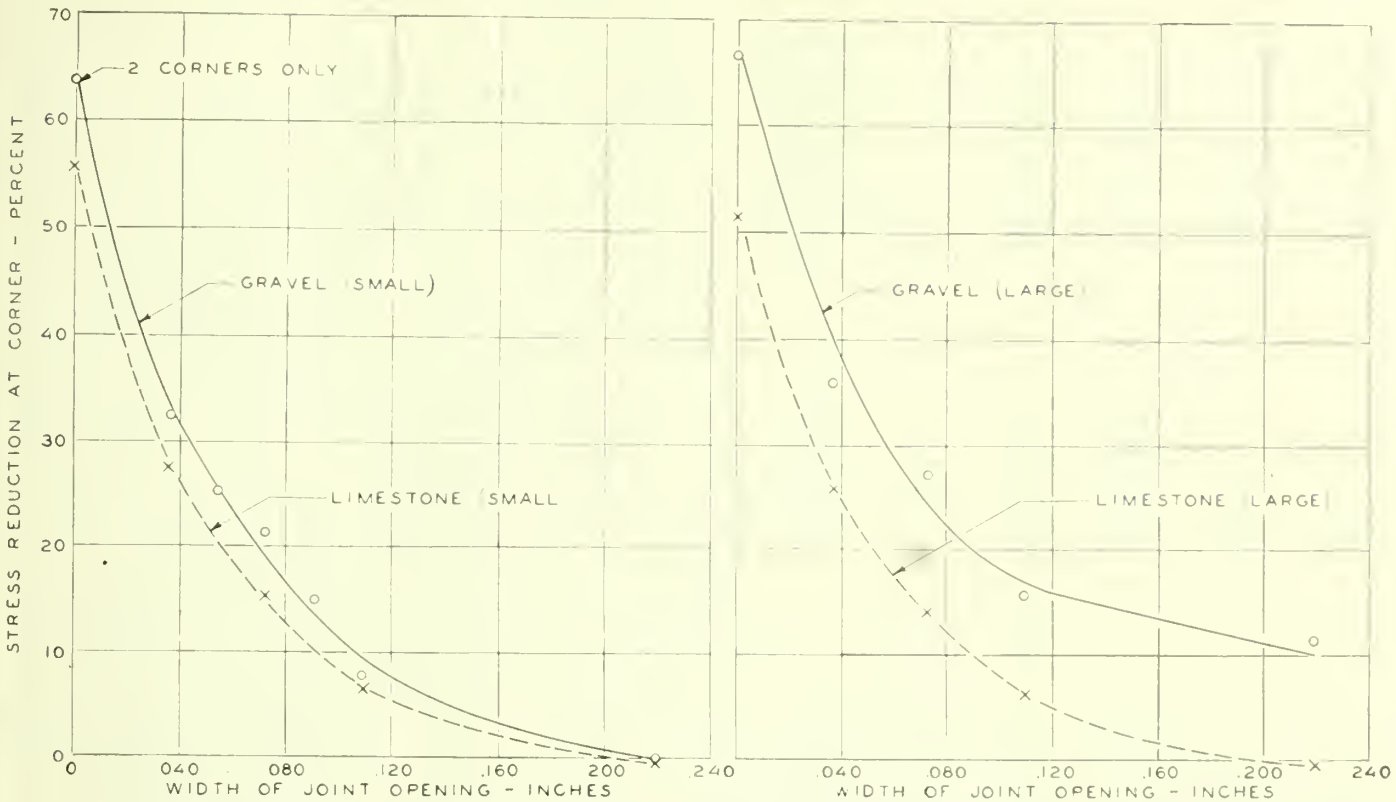

FIGURE 17.—EFFECT OF TYPE OF COARSE AGGREGATE ON THE RELATION BETWEEN JOINT OPENING AND STRESS REDUCTION AS DETERMINED WITH CORNER LOADING. EACH VALUE IS BASED ON THE MAXIMUM AVERAGE STRESS AS DETERMINED BY TESTS ON CORNERS 1 TO 4, INCLUSIVE.

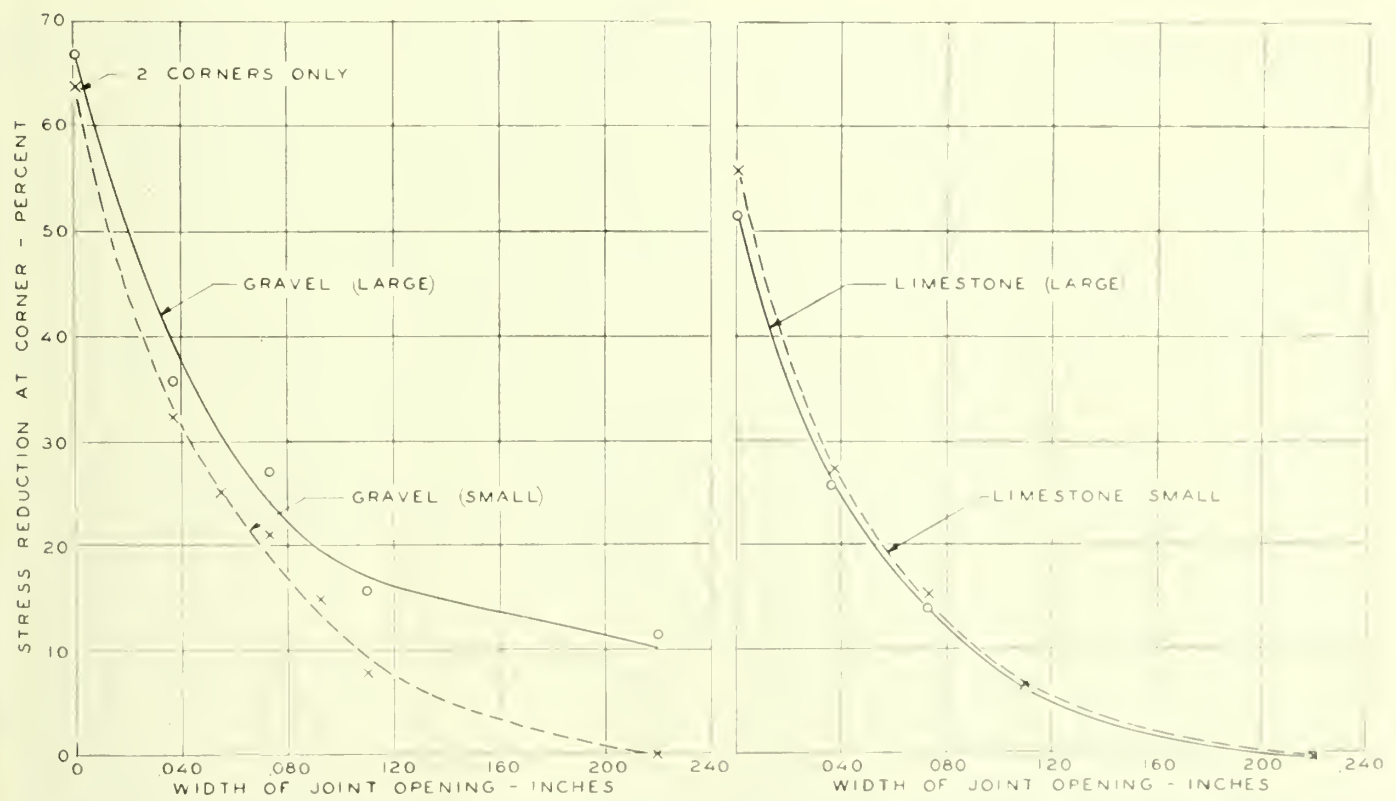

FIGURE 18.—EFFECT OF SIZE OF COARSE AGGREGATE ON THE RELATION BETWEEN JOINT OPENING AND STRESS REDUCTION AS DETERMINED WITH CORNER LOADING. EACH VALUE IS BASED ON THE MAXIMUM AVERAGE STRESS AS DETERMINED BY TESTS ON CORNERS 1 TO 4, INCLUSIVE.

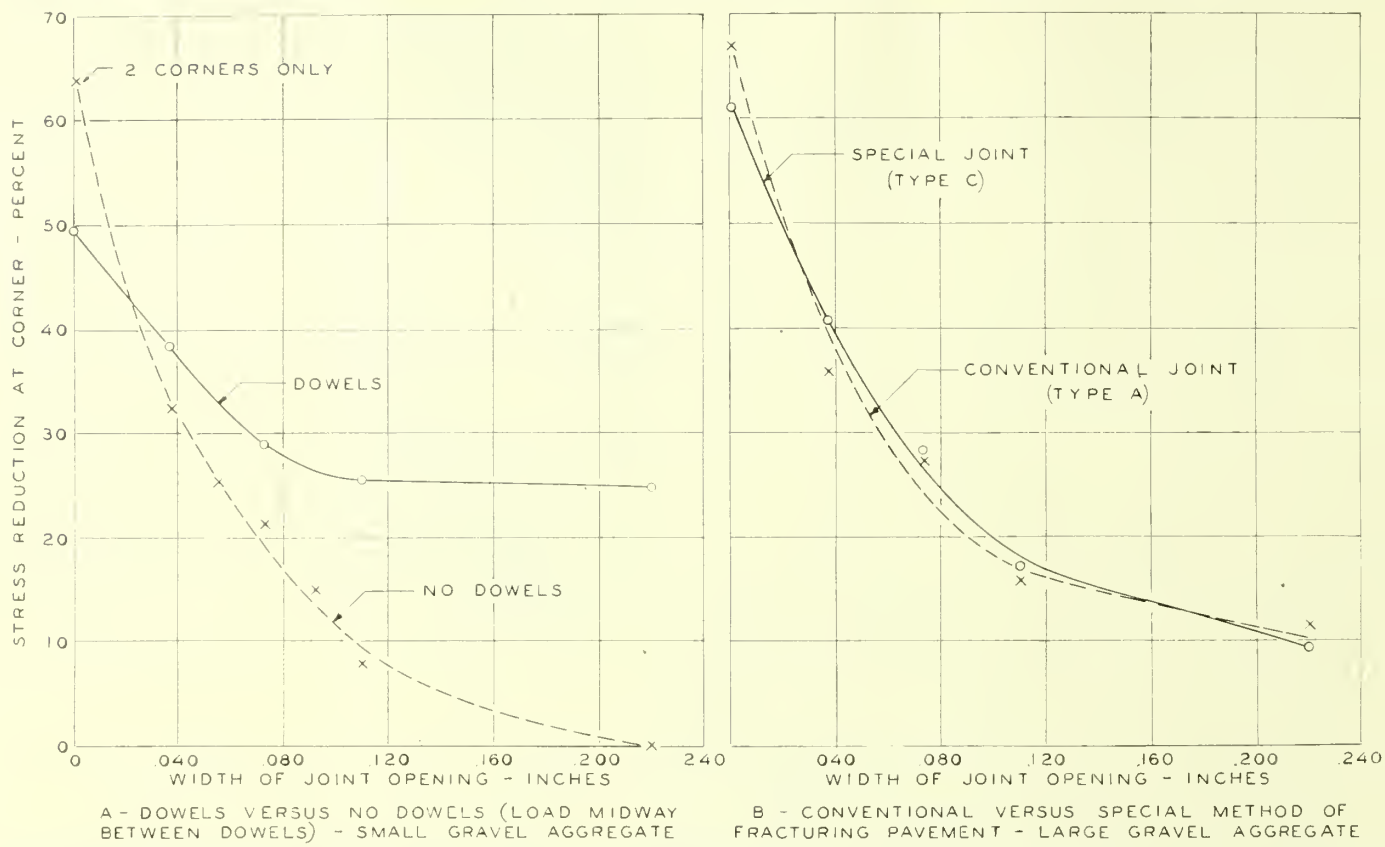

FIGURE 19.—EFFECT OF DESIGN DETAILS ON THE RELATION BETWEEN JOINT OPENING AND STRESS REDUCTION AS DETERMINED WITH CORNER LOADING. EACH VALUE IS BASED ON THE MAXIMUM AVERAGE STRESS AS DETERMINED BY TESTS ON CORNERS 1 TO 4, INCLUSIVE.

load that can be transferred by any joint is slightly less than 50 percent. Thus, with a load acting at a joint corner, the maximum reduction in stress that might be expected by load transfer is about 50 percent. However, if the joint is capable of exerting a moment which resists bending as well as transferring load through plain shear, it is possible that it might cause a reduction in stress greater than 50 percent of that for the free-corner condition. Apparently, the reductions in stress above 50 percent found at the zero width of joint opening were due to the resistance to bending caused by interfacial pressure at the joint.

The residual opening of joint No. 3 (large gravel aggregate) was 0.14 inch and that of joint No. 6 (large crushed-stone aggregate) was 0.02 inch when these joints were retested while under compression after completion of the scheduled tests. These separations had the effect of reducing the average corner stress-reduction value (joint under compression) of joint No. 6 from 67 to 25 percent and the value of joint No. 3 from 52 to 47 percent.

The relations presented in figures 17, 18, and 19 indicate that there was a sharp reduction in the effectiveness of the joints as the compression between the joint faces was released. Also that there was a progressive reduction in their effectiveness as the width of the joints were increased beyond 0.037 inch.

Figure 17 was prepared to show the effect of the type of coarse aggregate on the reduction in stress caused by aggregate interlock. It is indicated by this figure that the gravel gave a more effective interlock than the crushed stone. The difference between the two aggregate types was not great for the small-size coarse aggregate, but was appreciable in the case of the larger course aggregate.

In figure 18 is shown, for both types of coarse aggregate, the influence of the size of aggregate on the effectiveness of the aggregate interlock. It is observed that the large gravel gave a more effective interlock than the small gravel, but the size of the aggregate appeared to have little influence in the case of the crushed-stone aggregate.

It is believed that the reason for the more effective interlock shown by the gravel aggregate, particularly the large size, in these corner tests is the same as that discussed earlier in the presentation of the data for the edge condition of loading.

Comparisons are shown in figure 19 between similar weakened-plane joints with and without dowels and between the special weakened-plane joint, type C, and one of a conventional type. The coarse aggregate in the joints compared first (fig. 19, *A*) was small gravel while that in the other joints compared second (fig. 19, *B*) was large gravel.

It will be noted that at the zero width of joint opening (joint under compression), the stress reduction for the joint without dowels was greater than that of the joint with dowels. This may be the result of a difference in the effectiveness of the aggregate interlock in the two joints since the dowels have little or no opportunity to act while the joint is under compression or until some relative displacement between the two sides of the joint has developed. The stress reduction of the doweled joint exceeds that of the comparable un-

doweled joint at openings greater than approximately 0.02 inch, the difference being considerable at the large openings.

There appears to be little or no difference in the effectiveness of the aggregate interlock in the special weakened-plane joint, type C, and the conventional weakened-plane joint, type A.

EFFECTIVENESS OF AGGREGATE INTERLOCK INFLUENCED BY SMALL CHANGES IN JOINT WIDTH

The percentage of stress reduction at the four individual corners of each joint are shown for three different joint openings in figure 20. With the joint in a closed condition (under compression) the stress reduction values at the four corners were found to be quite uniform at all of the joints, although at joint No. 6 there was somewhat more variation than at the corners of the other five. However, as the joints were opened, appreciable variations were found in the amount of stress reduction among the four corners of each of the individual joints. At the 0.037-inch opening, the maximum variations occurred at joints Nos. 2, 5, and 6 where the stress-reduction values at the individual corners ranged as much as 25 percent and the amount of stress reduction at the corner showing the maximum was more than double that at the corner showing the minimum. The doweled joint No. 4 showed the most uniform stress reduction, the range in reduction values being approximately 5 percent.

At the 0.073-inch opening the stress reductions at some of the individual corners of several of the joints were so low as to indicate that, at this opening, aggregate interlock is not dependable. A possible exception to this is joint No. 3 with a large gravel aggregate. However, this joint would, without question, lose its effectiveness at only a small increase in opening or, perhaps, by the action of traffic gradually destroying the projecting aggregate.

The doweled joint at the 0.073-inch opening showed the greatest uniformity in percentages of stress reduction, 23 to 34 percent, and the highest average stress reduction, 29 percent. A special test was made of this joint at an opening of 1 inch and it was found that, at this opening, the stress reduction at the four corners ranged between 20 and 25 percent and averaged 22 percent. Thus, even at an opening of 1 inch, this joint not only maintained a high degree of uniformity in stress reduction but the difference in the average of the reduction values was only 7 percent when the width of the joint was changed from 0.073 to 1 inch. The efficiency of this doweled joint in controlling the corner stresses was not as high as might be desirable on the basis of perfect transmission of shear, but this was probably due to the fact that the dowels were not of sufficient diameter to be highly effective in an 8-inch pavement.

In the preceding discussion it was assumed that a joint acting in shear only should, if it is 100 percent effective, cause a reduction in the critical load stresses at the corners of 50 percent. It might be contended that it is only necessary for a joint to cause a sufficient reduction in the critical corner stresses to make them equal to those caused by an equivalent load acting at the interior of the slab.

A comparison of the average of all of the critical free-corner strains with the average of all of the critical interior strains obtained from the six sections of this study indicated that a reduction in the corner strains

FIGURE 20.—VARIATION IN THE AMOUNT OF STRESS REDUCTION AT INDIVIDUAL CORNERS 1 TO 4, INCLUSIVE, AT WEAKENED-PLANE JOINTS. THE JOINT-CLOSED CONDITION IS THAT CREATED BY A COMPRESSIVE FORCE OF 120,000 POUNDS APPLIED AT ONE END OF THE SECTION.

of approximately 27 percent is necessary if the load stresses at the interior and corners of the slab are to be equalized. Thus, if it is desired to reduce the corner stresses only enough to make them approximately equal to the interior stresses, it might be considered that all of the joints with gravel aggregate were sufficiently effective at an opening of 0.037 inch. At this opening the stress reduction of a number of the individual corners with crushed-stone aggregate fell appreciably below 27 percent.

At the 0.073-inch opening the stress reduction at one or more of the individual corners was appreciably less than 27 percent in all of the joints except joint No. 4, which contained dowels.

It is concluded from the preceding data on the corner loading tests that: (1) on the whole, all of the joints were more effective in reducing critical corner stresses than they were in reducing critical edge stresses; (2) in the presence of interfacial pressure and small residual openings, all of the joints were not only extremely effective but a high degree of uniformity of stress reduction was indicated among the individual corners; (3) in the presence of interfacial pressure and a relatively large residual opening (0.14 inch) the reduction in the critical corner stress was sufficient to make the stress approximately equal to that caused by a load of the same magnitude acting at the interior;

(4) without interfacial pressure, all of the joints depending solely upon aggregate interlock to reduce critical stresses indicated a wide variation in the stress-reduction values; (5) the effectiveness of aggregate interlock was improved by roughness of the joint faces; and (6) the dowels improved the uniformity of stress reduction among the individual joint corners and, even at large openings, reduced the critical corner stress to a value approximately equal to that caused by a load of the same magnitude acting at the interior.

SIGNIFICANCE OF THE RESULTS

In concrete pavements the amount that closely spaced contraction joints may open during the winter when the concrete is in a contracted condition depends primarily upon the following factors: (1) The temperature of the concrete at the time it hardens; (2) the thermal coefficient of the concrete and the range in temperature to which the pavement is subjected; (3) the spacing of the joints; (4) the amount of shrinkage that occurs when the concrete hardens; and (5) the change in volume of the concrete due to seasonal moisture variation.

In a pavement laid without expansion joints, the expansion beyond the length at which it hardens is restrained; hence the maximum change in the widths of the contraction joints, due to volumetric changes in the concrete, is dependent upon the changes which occur from the time the pavement hardens to the time it reaches its minimum length. In a pavement with sufficient expansion space to permit full expansion, the maximum change in the width of the joints is dependent upon the volumetric changes during the annual cycle of moisture and temperature variations. Thus, to compute the maximum opening of the joints it is necessary in the case of pavements without expansion joints to assume the hardening temperature of the concrete and the minimum temperature that may occur. For pavements with expansion joints, it is necessary to assume the maximum and minimum temperatures that may occur in the concrete. It should be possible, with the help of published data on the temperature of concrete pavements, to assume these temperatures with reasonable accuracy. A number of determinations have been made of the thermal coefficient of concrete and while it varies with different aggregates it has been found to average approximately 0.000005 per degree Fahrenheit. Unless the spacing of the joints is known, it will be necessary to assume a value for purposes of computation.

There is a limited amount of data available on the shrinkage of concrete pavement slabs during the hardening period and, also, the change in length caused by seasonal moisture variations. In a study in Minnesota (*2*) measurements were made of the shrinkage in a number of slabs over a period of several years. It was found that during the early hardening period the amount of shrinkage was equivalent to that which would be caused by a reduction in the average temperature of the concrete of approximately 16° F., but that the amount of shrinkage increased appreciably later. Observations have been made of the changes in length of several full-size concrete pavement slabs from seasonal moisture variations. These observations indicate that the change in length depends upon the weather conditions, but is generally equivalent to that which might be caused by a change in the average temperature of the concrete of 20° to 30° F. Thus, within the limits of these data, it appears that the seasonal change in length of a pavement slab caused by seasonal moisture variation, from summer to winter, is approximately equal to the shrinkage which occurs when the concrete hardens.

To permit the measurement of the opening that is expected to occur later at contraction joints in concrete pavements, it is necessary to make the basic measurement before the joints fracture. This was done in the Michigan study (*1*). Measurements were made in the central part of a section of road without expansion joints and with contraction joints spaced 20 feet apart. During the second winter, that is, the first winter after the pavement had expanded to its maximum length, the average joint opening was 0.06 inch. The opening should be correspondingly greater or less for other contraction joint spacings.

EFFICIENCY OF WEAKENED-PLANE JOINTS MAY BE IMPAIRED WITH TIME

In pavements laid with expansion joints there is a tendency for contraction joints to open progressively until all available expansion space is dissipated. Because of this, the actual opening of the joints may, after a period of time, greatly exceed the computed openings or those observed during the first or second winter after the pavement is laid. In pavements without expansion joints there is little opportunity for progressive changes in the widths of the contraction joints. It has been observed, however, that where contraction joints are spaced at close intervals fracture may occur at some of the joints soon after the pavement is laid, yet be delayed as much as a year or more at others. Under these conditions some of the joints would open more and others correspondingly less than the normal amount. Should the open joints become filled with foreign matter they are apt to maintain a large part of this abnormal opening throughout the life of the pavement.

It was shown by the tests of the weakened-plane joints in this investigation that aggregate interlock was effective in stress control when the joints were closed or under compression, but that it was not dependable when the joints were open 0.037 inch or more, irrespective of the type or maximum size of the aggregate in the concrete. Thus, it must be concluded that aggregate interlock cannot be depended upon to give effective stress control throughout the full yearly temperature cycle in pavements with contraction joint spacings such that the joints can open an amount greater than approximately 0.04 inch.

Aggregate interlock may, during the early life of the pavement, be of some benefit in reducing the critical load stresses at weakened-plane joints which open less than this amount. The elimination of expansion joints might be expected to increase the length of the period over which the joints would retain their initial effectiveness, but it is doubtful if it would make them permanently effective. It has been found that joints which permitted a relative deflection, between the two sides of the joint, of approximately 0.008 inch or more under critical loads were ineffective in stress control; so that a joint to be effective must become fully engaged at very small deflections. Heavy, high-frequency traffic causes a severe hammering action in weakened-plane joints when they are open even small amounts and it is only natural to expect that, over a

period of several years, this hammering action would break down the aggregate interlock sufficiently to permit the small relative deflection that makes the joint ineffective in stress control.

In pavements where stress control at the joints is desirable it, therefore, appears necessary to use some more effective device for load transfer. The doweled joint tested in an open condition in this investigation, although more effective than the plain weakened-plane joints, did not show the structural efficiency expected. Other tests on doweled joints made by the Public Roads Administration indicate that ¾-inch dowels at 12-inch intervals are quite effective in stress control in slabs of 7 inches or less in thickness. This suggests that the size of the dowels should be progressively increased with increase in the thickness of the pavement.

PUMPING AND FAULTING AT CONTRACTION JOINTS DISCUSSED

One of the most serious conditions that has developed in concrete pavements in recent years is faulting and pumping at expansion joints, open cracks, and open weakened-plane joints. This condition is probably aggravated by a greater frequency of heavy loads on the pavements during the war period than has been common in the past. Faulting generally follows pumping, but may occur without pumping. The type of faulting not associated with pumping has been found on subgrades of both fine-grained and granular soils, but seems to be most common and serious with silt and clay subgrade soils. Pumping is definitely associated with fine-grained subgrade soils and observations indicate that the water which is pumped to the surface from beneath the slabs is largely surface water that has leaked through the joints, open cracks, and along the edges of the pavement. Hence, defective sealing of joints and cracks is a condition contributing to pumping and the type of faulting associated with it.

Aggregate interlock at weakened-plane joints and cracks that are not permitted to open too widely seems to have been generally effective in preventing faulting of the type not associated with pumping. In pavements laid without expansion joints and closely spaced contraction joints (15 to 20 feet) aggregate interlock probably will be effective in preventing serious faulting of this type. If conditions are such as to permit an appreciable progressive opening of the weakened-plane joints, however, it is thought that an additional means for load transfer should be provided.

Adequate load transfer acts in two ways to prevent or reduce the amount of pumping and the accompanying faulting at joints in concrete pavements. First, it reduces the total deflection that occurs when the loads pass over the joint, thus causing a reduction in the amount of water and soil pumped to the surface with each repetition of load. Second, it reduces the relative deflections of the two slab ends when a load passes over the joint thus helping to preserve the seal of the joints and prevent the infiltration of surface water. Tests have shown that the total deflections at joints may be reduced nearly 50 percent and the relative deflections nearly 100 percent by adequate load transfer installations.

It has been found that aggregate interlock may be very helpful in reducing the total deflections and relative deflections at joints if they are not allowed to open too widely. However, it must be recognized that heavy, high-frequency traffic may in time seriously reduce the effectiveness of aggregate interlock. From load-deflection studies on slabs not subjected to traffic, it has been observed that the performance of weakened-plane joints with dowels was definitely better than those without dowels. Also, it may be expected that the doweled joints will retain their effectiveness at wider openings and over a longer period of time.

The effectiveness of load-transfer devices in preventing pumping and the accompanying faulting naturally varies with the character of the devices. Field observations of pavements indicate that, in many instances, where the conditions are not too severe, plain round ¾-inch dowels spaced at close intervals have been effective in preventing or minimizing pumping and faulting. These same observations indicate that all types of load-transfer devices are not equally effective for this purpose.

Since aggregate interlock is of definite value in preventing or reducing the amount of pumping and faulting in weakened-plane joints when they are not allowed to open too widely, it is desirable that some provision should be made in the design of the pavement to limit the opening of the joints. This involves the adjustment of the spacing of such joints to limit the magnitude of the seasonal opening from temperature and moisture variations in the concrete, and the limiting of available expansion space to prevent appreciable progressive opening of the intermediate joints, either by actually eliminating the expansion space or by using an expansion joint filler that offers considerable resistance to compression.

It is desirable that further studies be made to determine the effectiveness of aggregate interlock and other methods of load transfer in preventing pumping and faulting and the conditions under which other corrective measures may be necessary.

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DISINTEGRATION OF BRIDGE CONCRETE IN THE WEST

Reported by F. H. JACKSON, Principal Engineer of Tests, Division of Physical Research, Public Roads Administration[1]

A report giving the results of an inspection of numerous structures located in certain areas of Wyoming, Oregon, Washington, and California, made in connection with a study of the causes of the disintegration of concrete in these areas and including suggestions for revisions in specifications to govern future work.

FOR SOME YEARS it has been evident that concrete bridges located in certain areas of the west are not giving as satisfactory service insofar as the durability of the concrete is concerned as similar structures in other areas built under identical specifications and subject, in general, to the same degree of engineering supervision during construction. Reports by representatives of the Public Roads Administration, as early as 1935, called attention to disintegration which was starting on some of the bridges in the Jackson Hole area just south of Yellowstone Park. These bridges were at that time only about 4 years old. They now show evidence of advanced disintegration. Moreover, other bridges much more recently constructed and located within the park are showing initial evidence of distress of about the same type as was first noted on the older bridges.

Mr. G. S. Paxson, bridge engineer of the Oregon State Highway Commission, in a letter dated June 17, 1943, described several distinct types of disintegration of concrete which had developed about the same time (1934–35) on certain bridges located in eastern Oregon, whereas bridges of the same design and built under the same specifications, but located in the western part of the State, were showing no signs of trouble. Mr. Bailey Tremper, materials engineer, Washington Department of Highways, has also called attention to evidences of disintegration of concrete in certain areas of that State. In a paper published in June 1941 (*6*)[2] he discussed deterioration of concrete in eastern Washington and in the general area of Mount Rainier National Park. The trouble in the park area was identified by Tremper as probably due to a cement alkali-aggregate reaction, a particular type of concrete failure which Mr. T. F. Stanton, materials and research engineer for the California Division of Highways described about 1 year earlier as prevalent in certain areas in southern California (*4*). Numerous articles on the subject have appeared since.

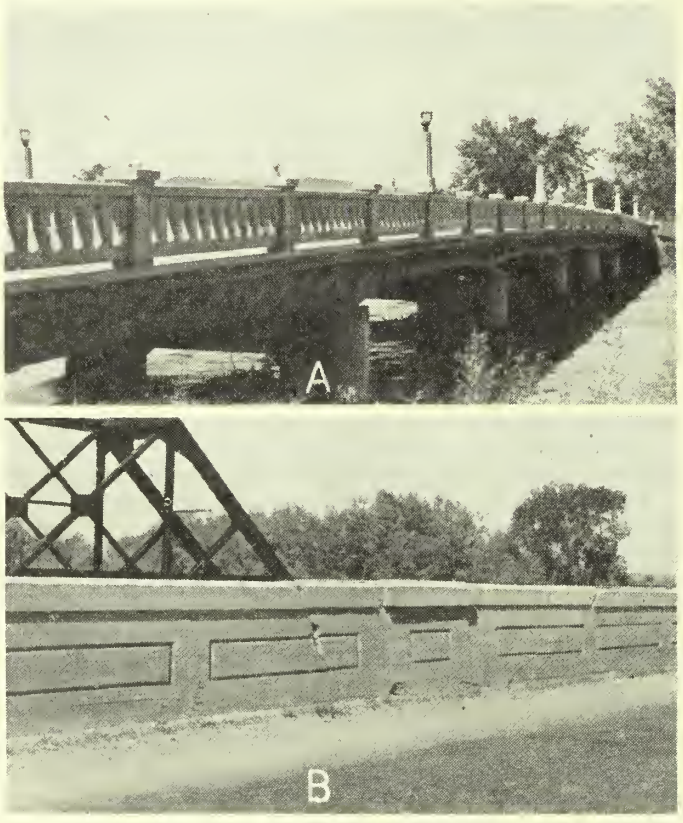

FIGURE 1.—(*A*) OLD CONCRETE BRIDGE IN WYOMING IN GOOD CONDITION AFTER 22 YEARS. THE BRIDGE WAS BUILT IN 1921. (*B*) AN EXAMPLE OF TYPE 1 DETERIORATION IS A BRIDGE IN WYOMING BUILT IN 1921. THE CONCRETE IS IN GOOD CONDITION EXCEPT FOR CRACKS DUE TO IMPACT FROM COLLIDING VEHICLES.

From the above it is evident that failures of concrete in these areas cannot be associated with the work of any particular agency—bridges built by the Public Roads Administration as well as those constructed by the States showed evidence of distress under certain conditions. Furthermore, it should be emphasized that even in the areas where trouble has occurred not all of the bridges are showing distress. There are many good bridges in these areas, particularly among the older structures (fig. 1, *A*).[3] However, the defects which have developed, in some cases after only 3 or 4 years, are sufficiently serious to warrant a very intensive search for the causes in order that steps may be taken to avoid these troubles in the future.

[1] Three separate parties were organized to make these inspections. In addition to the writer, they included the following:

(1) For the inspections at Kimball, Nebr., and in Wyoming including Yellowstone Park: B. W. Matteson, district engineer, District 3, Denver; H. R. Angwin, senior highway bridge engineer, Regional Office, San Francisco; A. V. Williamson, senior highway engineer, District 3, Denver; W. D. Ross, materials engineer, District 3, Denver; H. S. Meissner, engineer in charge cement laboratory, Bureau of Reclamation, Denver, and M. A. Ver Brugge, materials engineer, Wyoming State Highway Department.

E. H. Cowan, highway engineer, District 3, in charge of Public Roads work in Yellowstone Park, accompanied the party through the park. I. E. Russell, chief materials engineer, Wyoming, accompanied the party to Kimball, Nebr., in place of Ver Brugge.

(2) For the inspections in Oregon and Washington: G. S. Paxson, bridge engineer, Oregon State Highway Commission, Salem; Bailey Tremper, materials engineer, Washington State Department of Highways, Olympia; H. R. Angwin, senior highway bridge engineer, Regional Office, San Francisco, and R. M. Schwegler, materials engineer, District 1, Portland.

Mr. Stephenson, assistant bridge engineer, Oregon, accompanied the party during the inspections in Oregon only.

(3) For the inspections in California: T. E. Stanton, materials and research engineer, California Division of Highways, Sacramento; Fred Klein, senior highway bridge engineer, Regional Office, San Francisco, and D. J. Steele, materials engineer, District 2, San Francisco.

The itinerary for Wyoming and the Yellowstone Park was arranged by Matteson, for Oregon and Washington by Paxson and Tremper, and for California by Stanton. Inspections in Wyoming and the Yellowstone Park were made during the period June 12 to 20, inclusive; those in Oregon and Washington, June 23 to 30, inclusive; and those in California, July 4 to 11, inclusive, all in 1944.

[2] Italic numbers in parentheses refer to the bibliography, p. 111.

[3] The photographs accompanying this report were selected to illustrate as clearly as possible the various types of failure that were observed during the inspection. In addition to those taken by the writer, photographs taken by the following members of the inspection parties are included: Messrs. W. D. Ross, A. V. Williamson, H. S. Meissner, Bailey Tremper, T. E. Stanton, and R. M. Schwegler.

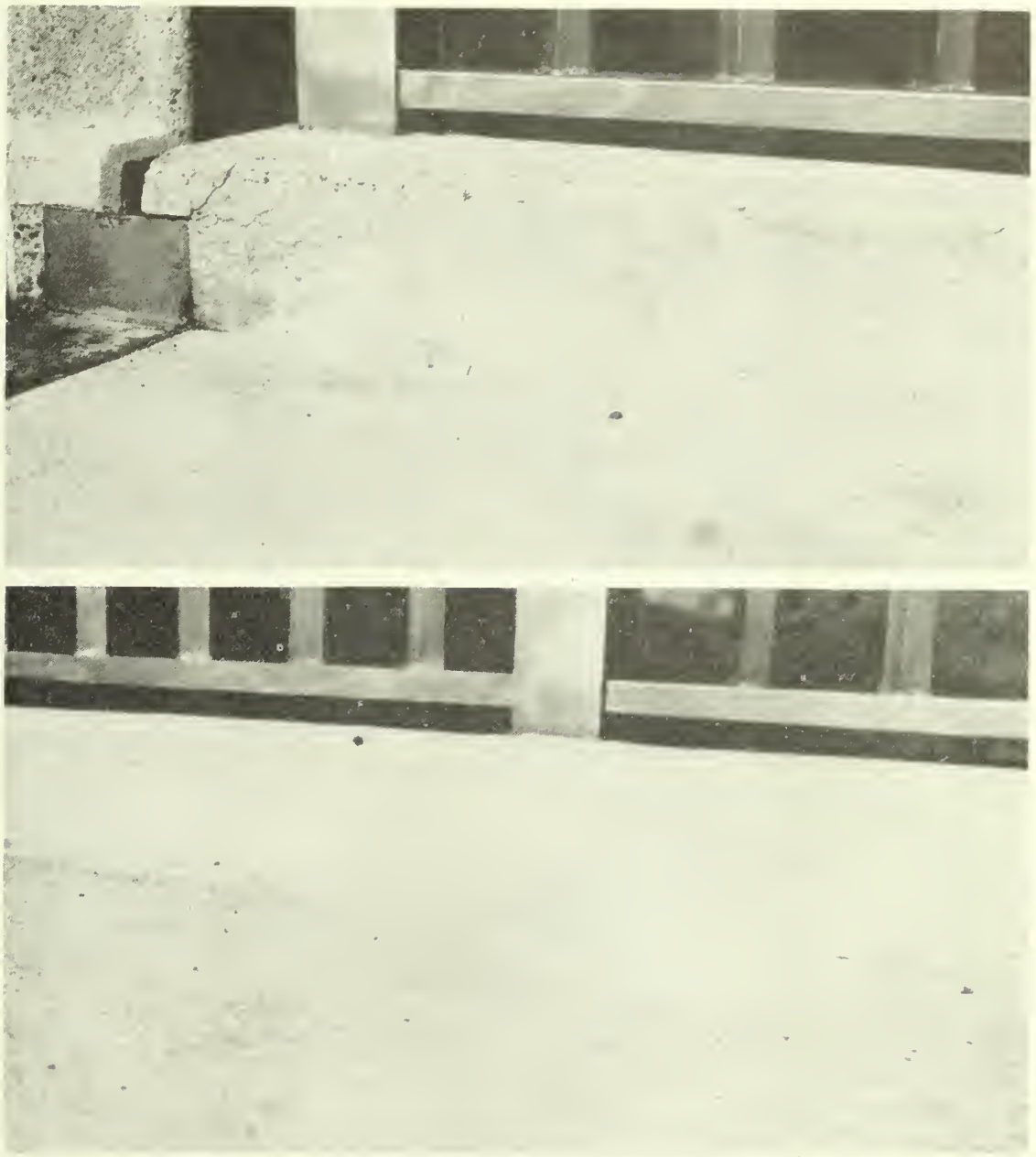

FIGURE 2.—FIRST STAGE OF TYPE 2 DETERIORATION ON A BRIDGE IN YELLOWSTONE PARK. FINE CRACKS APPEAR AT JOINT AND ALONG CURB.

The writer was assigned to make an inspection of certain bridges in and near Yellowstone Park and to confer with field engineers regarding the problem. Since the Public Roads Administration is actively cooperating with both Oregon and Washington in an effort to determine the causes of concrete failures in the Pacific Northwest, it was felt that the inspections should be extended to include bridges in these States. Arrangements to this end were made as well as arrangements to visit a number of structures in southern California which show evidence of failure due to cement alkali-aggregate reaction.

It is believed that because of the similarity of the problems the observations made during the inspection of structures in Oregon, Washington, and California, as well as the conclusion reached and corrective measures proposed, can appropriately be included in the report on the Yellowstone Park problem. In the following discussion it is proposed first, to describe the types of failure observed on the trip; second, to summarize the observations made in each of the areas visited; third, to discuss certain specific matters which may have a bearing on the problem, including construction variables, modern versus old-fashioned cements, air entrainment, alkalies, etc., and fourth, to indicate the corrective measures which should be adopted.

FOUR TYPES OF DETERIORATION OBSERVED

During the course of these inspections several distinct types of concrete deterioration were observed. These may be classified roughly as follows:

Type 1. Deterioration due to gradual or normal weathering.—This is indicated usually by slight surface erosion and pitting, rounded corners, etc. Many old structures also show cracks due to settlement and

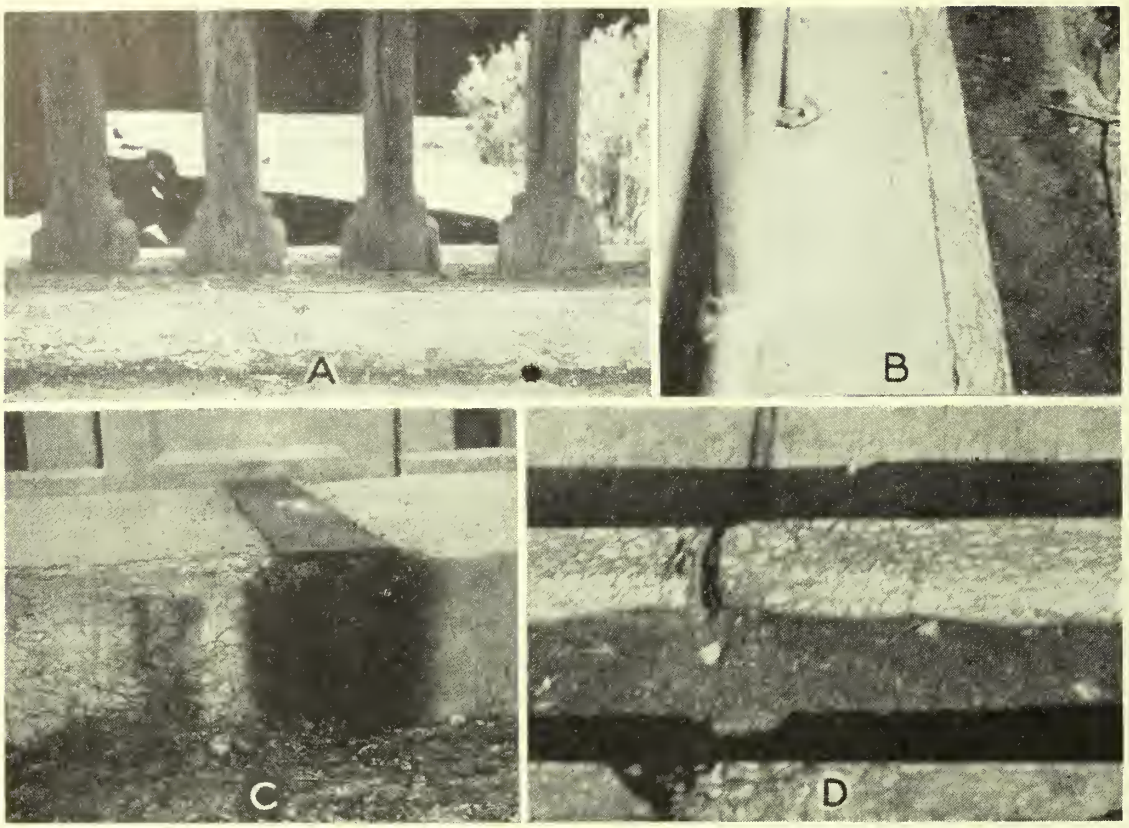

FIGURE 3.—EXAMPLES OF INTERMEDIATE STAGE OF TYPE 2 DETERIORATION IN (*A*) OREGON, (*B*) WYOMING, (*C*) ILLINOIS, AND (*D*) WYOMING. THE CRACKS ARE FILLED WITH DARK DEPOSIT SOMETIMES CALLED D-LINES.

impact from colliding vehicles (fig. 1, *B*). The concrete has a good ring under the hammer and a good sharp chip may be obtained. This condition is evidence of durable concrete, especially when found on structures subject to severe natural weathering. Concrete of this character was noted on many of the older bridges.

Type 2. Deterioration due to accelerated weathering.—This type of failure is all too common, particularly in areas subject to severe frost action. It is usually evidenced first by the formation of fine cracks on the surface of exposed members such as curbs, handrails, end posts, tops of retaining walls, wing walls, etc. They usually appear first on surfaces adjacent to construction joints or at cracks and other points where water can enter (fig. 2, *A*). They also tend to form along the edges of thin members such as curbs and handrails (fig. 2, *B*). The cracks are ordinarily close to and parallel to the edge and are usually filled with a dark-colored deposit, probably calcium carbonate (fig. 3). Concrete so affected has little strength and can be broken easily under the hammer. The matrix has a dull chalky appearance in sharp contrast to the dense, compact, bluish-gray matrix usually found in good concrete. Progressive and rapid disintegration usually follows the appearance of these D-lines as they are frequently called.

As might be expected distress of the type 2 variety, if it develops at all, usually starts in the relatively thin members of the superstructure. They are the members most directly exposed to weathering. The sections are frequently thin, necessitating the use of a rather fine graded, coarse aggregate which, in connection with the fact that these members usually contain considerable reinforcing steel, tends to result in the use of a higher water content than in the heavier members, even though the cement content in bags per cubic yard may be exactly the same. This raises a question as to whether thin sections should be used where the concrete will be subjected to severe weathering.

D-line cracking of the above type should not be confused with the familiar crazing cracks which often appear on highly finished smooth surfaces and which are, no doubt, due to surface shrinkage. D-lines on the other hand are probably the result of internal expansion resulting from failure due to alternate freezing and thawing. So far as the writer is aware, they are not found in areas where freezing does not occur.

Type 3. Deterioration due to salt scaling.—This type of surface defect is the result of using calcium or sodium chloride for ice removal. Pavements and sidewalks so treated, especially if the concrete is new, will almost invariably scale badly, the mortar surface being, in many cases, completely removed during the first winter (fig. 4). A specific remedy for this condition has been found in air entrainment, which will be discussed later.

Type 4. Cracking due to abnormal expansion.—All concrete will expand or contract when subjected to changes in either temperature or moisture content. This property is recognized and provided for in the design of structures. Occasionally, however, a tendency towards abnormal expansion develops which cannot possibly be explained in the usual way. Failures due to abnormal expansion are characterized by the formation of relatively wide, open cracks of appreciable depth and usually roughly parallel to the longitudinal axis (fig. 5). Random or pattern cracks, also open and rather widely spaced, are frequently formed (fig. 6). Failure due to abnormal expansion may or may not be

FIGURE 4.—AN EXAMPLE OF TYPE 3 DETERIORATION ON A BRIDGE IN WYOMING, SHOWING SURFACE SCALING DUE TO THE USE OF SALT FOR ICE REMOVAL.

followed by ordinary weathering (type 2) depending upon the severity of exposure. For example, certain structures in southern California which showed characteristic cracking of this type as early as 1937 are still in about the same condition, indicating that expansion has ceased (fig. 5, *B*). In the absence of frost no further action has developed in these particular structures. On the other hand, there is considerable evidence from structures located in higher altitudes to indicate that failures which may have been started by abnormal expansion have been greatly accelerated by freezing and thawing, ultimate disintegration resulting from a combination of the two.

There is evidence to indicate that some of the earlier failures on certain bridges in eastern Oregon (all of which have been repaired) were due to the use of unsound cement. If this is true, there is no particular mystery about these failures, as it is well known that so-called hard-burned free lime may exist in cement that has been improperly burned and that the gradual hydration of this lime within the hardened concrete will cause disruptive expansion. Manufacturers of portland cement claim that adoption of the autoclave test for soundness has entirely eliminated trouble from this source. The test was not in use at the time these bridges were built.

The most serious form of delayed expansion observed cannot be attributed to unsoundness in either cement or aggregates. In all three of the areas visited, rather conclusive evidence of "cement-alkali aggregate" reaction was observed. This type of cracking is generally assumed to be due to internal expansion resulting from a reaction between the alkalies (sodium and potassium oxides) in the cement with certain siliceous constituents of the aggregates. In certain cases opaline silica has been identified as the reactive material in the aggregate. In other cases certain igneous rocks, mostly volcanic, have been identified as reactive. So far, evidence of such a reaction has been based almost entirely on observation of structures in service and on expansion tests of specimens containing high- and low-alkali cements in combination with aggregates of various types. Tests and experience correlate very well insofar as the identification of combinations which cause trouble is concerned. However, the actual mechanism of the action is not yet fully understood although various agencies (notably the Bureau of Reclamation and the Portland Cement Association) are working on this phase of the problem.

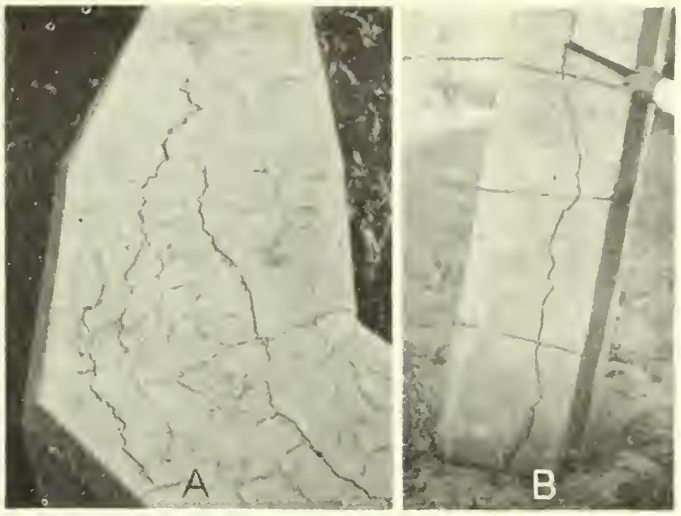

FIGURE 5.—EXAMPLES OF TYPE 4 DETERIORATION IN (*A*) NEBRASKA AND (*B*) CALIFORNIA IN WHICH THE MAIN PATTERN IS LONGITUDINAL.

Positive identification of the type or types of a particular failure is not always easy. This applies particularly to structures in which distress due to alternate freezing and thawing (type 2) may have been preceded by expansion due to an alkali-aggregate reaction (type 4). In such cases it is probable that the internal expansion due to chemical action may so weaken the concrete as to greatly lower its resistance to frost action. Some laboratory work recently reported by Tremper (7) indicates the possibility of such action. This theory also tends to explain the disturbing fact that some concrete structures built under modern specifications and under close engineering supervision, and which by all the rules should be durable, are not performing as satisfactorily as they should. However, the theory does not explain why many of the older structures located in the same areas are still in good condition. The cements used in the old structures (age 20 years or more) were probably as high in alkalies as were the corresponding cements made 10 or 12 years ago. The aggregates also were of the same general type. However, there were other differences in the cements which may explain the inability of some of the more recently constructed bridges to resist weathering as well as comparable structures built 20 or more years ago. This possibility will be discussed more fully later in the report.

During the course of the inspections, small hand samples of concrete were taken by the writer from a number of the bridges. These samples have been examined microscopically in the petrographic laboratory and the results reported in appendix I

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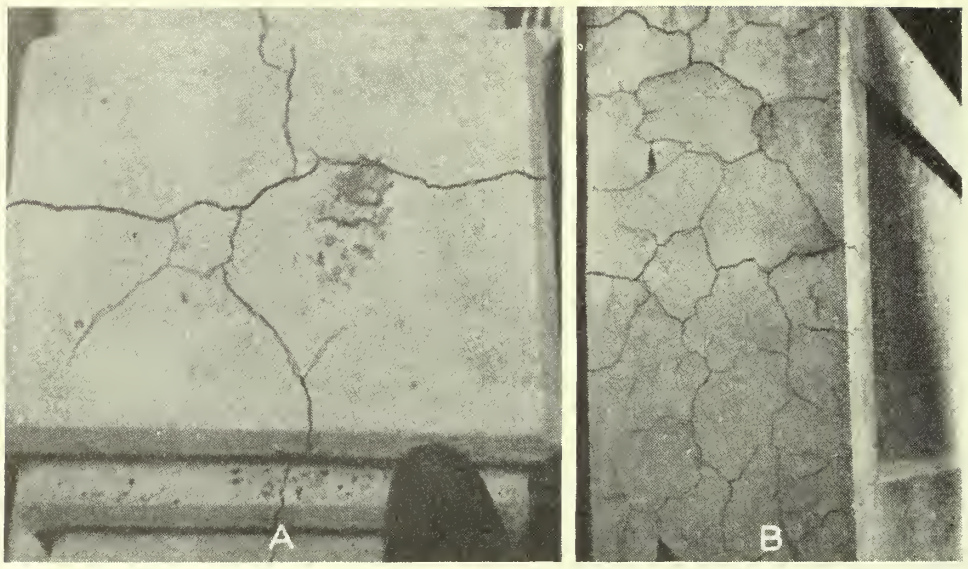

FIGURE 6.—EXAMPLES OF TYPE 4 DETERIORATION IN (*A*) NEBRASKA AND (*B*) WASHINGTON IN WHICH WIDE-OPEN RANDOM CRACKS DEVELOP.

FIGURE 7.—FAILURE OF A HANDRAIL IN WYOMING DUE TO ACCELERATED WEATHERING, AN EXAMPLE OF TYPE 2 DETERIORATION.

OBSERVATIONS IN WESTERN NEBRASKA, WYOMING, AND THE YELLOWSTONE AREA

During this inspection, some 40 or more structures located along the following route were observed. The party traveled from the Wyoming State line eastward on U S 30 to Kimball, Nebr.; on U S 30 westward from Cheyenne to Evanston, Wyo., thence north to the Jackson Hole area south of Yellowstone Park; thence through the park by way of Old Faithful, Madison Junction, Mammoth, Gardiner, Mont., northeast entrance road to Lamar River Bridge, the east loop road to Fishing Bridge; thence out of the park by the east entrance to Cody, Meeteetse, Thermopolis, Shoshoni, Casper, Douglas, Wheatland, and Cheyenne, Wyo.

Detailed discussion of the condition of the concrete in the structures inspected will not be attempted. It was impossible, in the limited time allowed, to make a detailed examination of all of the bridges or to take full and complete notes as to the condition of various portions of each structure. However, notes taken regarding the more important structures are in sufficient detail to give a very good idea of the condition of the concrete and related circumstances.

All four types of deterioration were observed on this particular trip. Several structures, particularly the older ones, showed no evidence of distress other than type 1. Among these may be mentioned an arch bridge over the Shoshone River just below the dam built in 1925; a bridge over the Big Horn River at Thermopolis built in 1922; a bridge over the North Platte at Casper built in 1921; a bridge over Deer Creek east of Glenrock built in 1921; and a bridge over the North Platte west of Douglas built in 1923. An old bridge over the Gibbon River, built by the War Department probably around 1913, was also in excellent condition considering its age. This bridge showed very little evidence of trouble except on one corner post, which was badly cracked. A sample from this post showed distinct evidence of an alkali-aggregate reaction (fig. 15, *A*).

Not all of the old bridges are in good condition, however. Mention should be made of two old bridges in the park built by the War Department, one over Can-

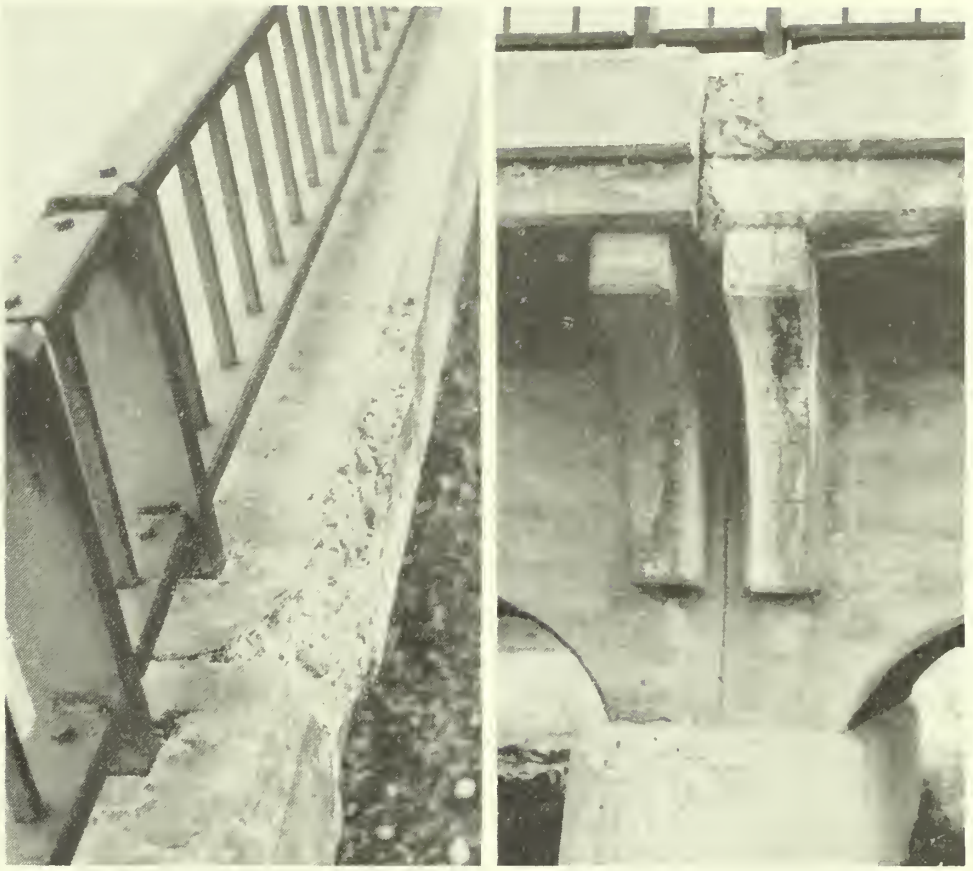

FIGURE 8.—AN EXAMPLE OF TYPE 2 DETERIORATION IN WYOMING DUE TO WEATHERING.

yon Creek and the other the Chittenden arch over the Yellowstone River. Both of these bridges show advanced disintegration due to weathering. There is no information as to when these bridges were built, but they are probably at least 30 years old.

Weathering (type 2) in all stages from initial D-line cracking to complete disintegration was observed in many structures both in and adjacent to the park as well as in other sections of Wyoming. Many examples were found which appeared to be simple cases of ordinary weathering. In other cases there was evidence that the effects of weathering may have been complicated by abnormal expansion of the type caused by an alkali-aggregate reaction.

Cases of simple type 2 weathering occurred in certain structures along U S 30 in Wyoming and in the superstructure of certain bridges in the Jackson Hole area, south of Yellowstone Park. In one of these structures, built in 1931, disintegration is confined almost entirely to the concrete handrails, end posts, and so forth. The apparent variability of the concrete, a rather typical condition, may be observed in figure 7, some of the rail being still in good condition. There is some evidence of abnormal expansion (other than might be caused by simple weathering) in the handrail of this structure. In this connection an investigation of the aggregates used in the bridge by the Bureau of Reclamation indicates that they are probably not alkali reactive. However, the bureau report does state that "certain of the weathered products could not be positively cleared of suspicion of susceptibility to alkali-aggregate reaction." It is proposed to investigate these materials further in the laboratory, including testing for expansion properties in mortar bars containing low- and high-alkali cements. Incidentally, reports indicate that, from the standpoint of physical soundness, the aggregates used in this structure were entirely adequate. The cement also was entirely satisfactory as measured by usual standards.

On another bridge built in 1931 and also located just south of the park, the distress appears to be entirely of the type 2 variety. There was no evidence of abnormal expansion due to chemical activity. In this structure there is considerable disintegration in the concrete brackets supporting the walk, in the curb supporting the steel handrail and in certain of the pier caps (fig. 8). Exposure conditions are extremely severe, temperatures ranging from 90° F. in the summer down to −50° F. in winter, with heavy precipitation and sufficient thawing in winter to produce a large number of alternations per year.

Within the park area proper, two bridges were of particular interest. One of these was built in 1939 and the other in 1940. Both bridges are showing signs of type 2 deterioration, evidenced by slight checking on the vertical surfaces of the curbs (fig. 2). The action is slight so far but is disturbing because the pattern is so familiar as the initial evidence of progressive disintegration.

Reference should also be made to a bridge over the Yellowstone River at Gardiner, Mont., built by the Public Roads Administration in 1930. This bridge is in excellent condition. Construction reports indicate that, with the exception of four cars, all of the sand was from a commercial pit located some distance from the project and that local sands were rejected due to low strength in mortar tests and an excess of obsidian.

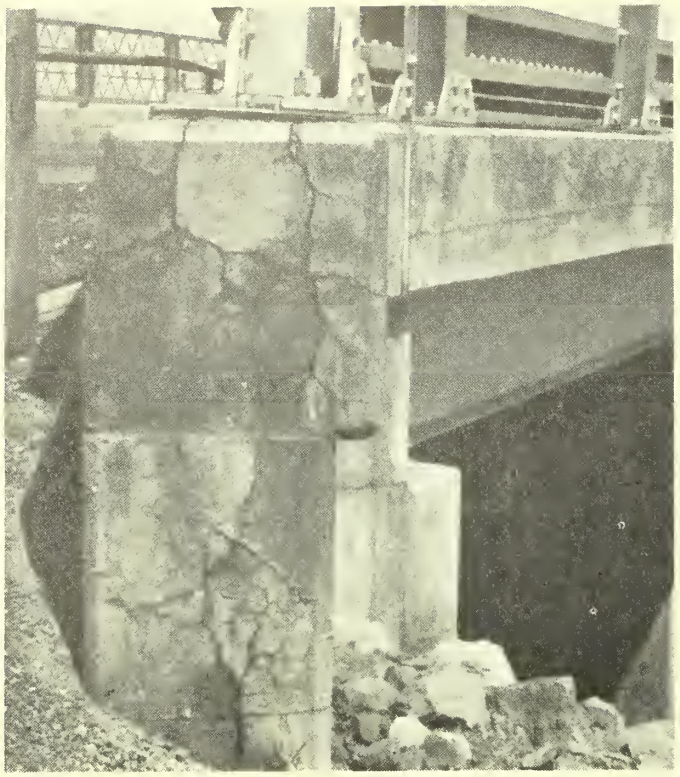

FIGURE 9.—PROBABLY AN EXAMPLE OF TYPE 4 DETERIORATION IN WYOMING SHOWING CRACKS IN THE END POST.

CAUSES OF DETERIORATION AND PREVENTIVE MEASURES DISCUSSED

The fact that some of the exposed concrete in the two park bridges as well as similar concrete in certain of the recently constructed bridges on the east approach road to the park is in the first stage of what appears to be type 2 weathering, is further evidence that even good concrete, according to usual standards, may be inadequate under this severe exposure. These bridges were built in accordance with the best engineering standards of the day, had adequate inspection, and certainly the high cement content used (6.5 sacks per cubic yard) would insure that water ratios did not exceed 6.0 gallons per sack. The concrete is not good enough and the question naturally arises—what can be done about it?

As to concrete in existing structures, it is strongly recommended that bridges showing this type of deterioration be waterproofed as soon as possible, using the linseed oil treatment which was proposed some years ago by the Portland Cement Association and successfully used by the Oregon State Highway Commission. In this connection, special attention is directed to the necessity for thoroughly repairing all affected areas by patching before the waterproofing is applied. This is the only way in which further progressive deterioration can be avoided. The methods used by the Oregon State Highway Commission for patching and waterproofing disintegrated concrete are outlined in appendix II.

For future work the writer recommends the use of an air-entraining agent in all exposed concrete in areas where severe frost action occurs. Considerations supporting this recommendation are discussed later.

In one of the Bureau of Reclamation reports relating to this problem, the opinion is expressed that the early failure of concrete in the bridges located south of the park may have been caused by the use of cements high in tricalcium aluminate (C_3A). The writer believes that the evidence that high C_3A content, per se, contributes to lack of durability is far from conclusive. Many of the older structures which are in excellent condition were made with cement high in C_3A. However, those older cements were not nearly so finely ground as modern cements. This point is also discussed later.

Evidence of type 3 deterioration (salt scaling) was confined to surfaces of pavements and sidewalks of structures where either sodium or calcium chloride had been used for ice removal. A particularly severe case was noted on a pedestrian underpass built in 1941. This trouble is so obviously due to salt treatment as to require little discussion except to call attention to the fact that air entrainment will very definitely increase resistance to this action. Resistance to salt action is highly important as the hazard from ice forming on concrete pavements is so great as to require the use of salts as a control measure even though there is danger of damaging the concrete. An air-entraining agent should be used in all concrete pavements which are apt to become hazardous through the accumulation of ice in winter.

Type 4 abnormal expansion was observed in pavements and grade-separation structures in western Nebraska, and in one structure on U S 30 in eastern Wyoming (fig. 9). Tests for expansion of mortar specimens containing the aggregates used in this bridge in combination with both high- and low-alkali cements are being made. The opinion has been expressed that the cracking in this structure may be due to differential expansion between the concrete and the steel handrail which is anchored in it, rather than to internal expansion of the concrete itself. This possibility must be examined.

Aside from the projects above noted and the old bridge in Yellowstone Park to which reference has been made, no other direct evidence of alkali-aggregate expansion was found during the inspection in District 3. However, as noted previously, the possibility of this reaction as a contributing factor in the disintegrating of these structures cannot be dismissed from consideration. It is quite possible, as Tremper's tests indicate, that the resistance of the concrete may have been weakened through the formation of fine expansion cracks caused by an alkali reaction. The intensity of the reaction in this area may have been considerably less than in the areas where it has been definitely identified, thus accounting for the absence of characteristic wide-open cracks. Aggregates in the Yellowstone Park area should be thoroughly examined with this possibility in mind.

OBSERVATIONS MADE IN OREGON AND WASHINGTON

During this portion of the trip approximately 120 bridges, and some pavements along the following route were examined.

In Oregon: From Ontario, north over U S 30 to Pendleton, thence south and west over U S 395 and U S 28 to Bend, thence south on U S 97 to Klamath Falls and returning to Bend; thence over the Cascades to Salem by way of the North Santiam River Highway; then to Portland.

In Washington: From Portland over U S 99 to Marys Corner; thence over No. 5 through the eastern portion of Mount Rainier National Park to Seattle; thence over U S 10 to Teanaway; thence through Blewett Pass to Peshastin; thence over No. 15 to the Berne undercrossing of the Great Northern Railroad,

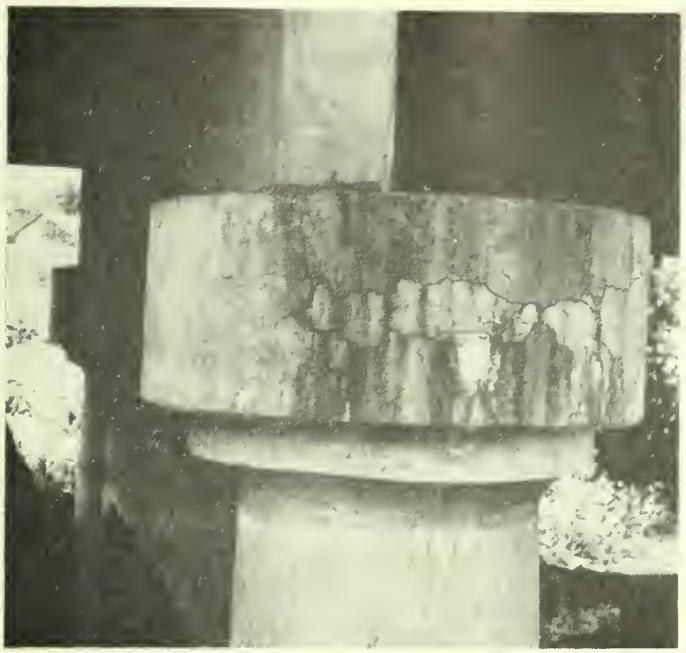

FIGURE 10.—POSSIBLY AN EXAMPLE OF TYPE 4 DETERIORATION IN OREGON SHOWING OPEN CRACKS IN THE PIER CAP.

thence to Wenatchee, thence easterly, following in general U S 10 alternate to Spokane; thence southwesterly through Ritzville, Lind, Pasco, Prosser, Goldendale, and along the Columbia River to Portland.

The routes followed in both States were chosen so as to include most of the areas in which trouble has been experienced. The fact that all of these areas lie either in or east of the Cascade Range, where climatic conditions are severe, and that no trouble has been experienced in the regions west of the mountains, indicates quite definitely that frost action is an important factor in the deterioration. The question is whether it is the only factor. There is evidence in both States to indicate that it may not be.

Most of the deterioration observed in eastern Oregon was of the familiar D-line or type 2 variety. Curbs and sidewalks of bridges were the most affected. Most of the precast rail pedestals were in good condition. Owing to the excellent bridge maintenance work of the Oregon Highway Department, most of the distress which caused so much concern in 1937 was not visible at the time of this inspection. This applies, among others, to several of the Burnt River bridges, to the Perry Arch over Grande Ronde River, to the overhead crossing at Meacham and to the Umatilla River bridge at Pendleton. The majority of the defects now showing are apparently the result of recent weathering on structures which have either been inadequately waterproofed or not waterproofed at all.

There is still evidence in places of the abnormal expansion which occurred in many of the bridges built with cement which may have been unsound. Some indications of continuing expansion were seen in one of the Burnt River bridges. The open cracks in the pier caps of another Burnt River bridge are indicative of abnormal expansion (fig. 10). As previously noted, an explanation of the excessive expansion of concrete containing a certain cement is that the cement in use at that time, due to improper manufacture, probably contained hard-burned free lime. However, according to State reports, this cement was also very high in

FIGURE 11.—POSSIBLY AN EXAMPLE OF TYPE 4 DETERIORATION IN OREGON SHOWING THE OPEN TYPE OF CRACKS.

alkalies, one report showing a value as high as 1.68 percent calculated as Na_2O. Therefore, the possibility of alkali-aggregate reaction cannot be definitely eliminated although it is understood that expansion bar tests run by the State on the aggregates used have not yet revealed unusual expansion. It is of interest to note that, of the 19 bridges reported to contain the high-alkali cement, 16 now show, or are reported to have shown, evidence of deterioration.

A view of wide pattern cracking, which may be type 4, is shown in figure 11. This is a grade-separation structure in central Oregon. The cracking seems to go beyond the familiar D-lines that are so prevalent on curbs and other thin sections.

Numerous structures were inspected in and around Klamath Falls. Many of these are in good condition, as for example, two bridges over the Link River, built in 1926 and 1931, and the Southern Pacific overcrossing north of Klamath Falls (1932). Others show extensive D-lines, principally in curbs and walks. This distress can probably be attributed to ordinary weathering. One of these bridges, built in 1930, shows evidence of expansion in longitudinal cracks in arch ribs. This bridge has been extensively repaired in an effort to prevent progressive disintegration. In spite of this there is some evidence of continuing disintegration in curbs and rails.

A recently constructed underpass built in 1939 is showing evidence of progressive disintegration. In this, as in many other structures, the disintegration is, in general, worse on the surfaces exposed to winter winds from the north. It is said that sleet frequently builds up on these surfaces while other surfaces are bare. This extreme exposure is particularly hard on concrete and it is quite possible that the disintegration

FIGURE 12.—AN EXAMPLE OF TYPE 4 DETERIORATION IN WASHINGTON SHOWING THE LONGITUDINAL TYPE OF OPEN CRACK.

in this area is entirely due to this cause. Here again the possibility of abnormal expansion due to chemical action within the concrete cannot be positively eliminated from the picture. Further investigations along this line must be made.

An interesting example of the possible effect of materials on the quality of concrete is found in the series of bridges along the North Santiam Highway, northwest of Bend. One of these, built in 1938, is showing serious disintegration, particularly on the railing. Sand and gravel from the Willamette River were used in this structure. On the other hand, several bridges along the same highway, built in 1934, in which local aggregates and a cement which has built up a very good service record, were used, are in very good condition. These materials are being studied in the co-operative investigation now under way.

MANY OLD STRUCTURES FOUND IN GOOD CONDITION

Many of the Oregon structures were in excellent condition. Among these were a number of old bridges on the old Oregon Trail, U S 30, between Ontario and Pendleton. These include Jacobson's Gulch Bridge, the overpass at Unity, an old bridge over the Grande Ronde River, and the overhead crossing at Glover. All four were built in 1922. The fact that so many of the old bridges are in good condition whereas many structures built much more recently are showing deterioration is further indication of the trend found on the inspection trip in District 3.

Considerably more of the wide-open type of cracking which is associated with abnormal expansion was observed in Washington than in Oregon. For example,

FIGURE 13.—PROBABLY EXAMPLES OF TYPES 2 AND 4 DETERIORATION IN WASHINGTON, SHOWING WELL-DEVELOPED PATTERN CRACKS.

in the area immediately adjacent to Mount Rainier National Park, disintegration of this type has been definitely identified as due to the use of aggregates from the Cowlitz and White Rivers in combination with cements high in alkalies. Bridges in the same general vicinity in which the same aggregates were used in combination with cements low in alkalies are in good condition. The condition of these bridges at the present time is about as described by Tremper in 1941, except that deterioration has progressed to some extent. In addition to deterioration in the superstructure, expansion cracks are now evident in the arch ribs and in various parts of the substructure of several of these bridges. Figure 12, showing longitudinal cracking in a rail coping, and figure 6, *B*, showing characteristic wide-open cracking in an end post, are typical.

Wide-open cracks indicative of expansion cracking were found in the northeast rail of a bridge on U S 99 over the Toutle River north of Kelso. This is the only bridge west of the mountains showing disintegration that came to the attention of the writer.

The inspections in central Washington included several bridges southwest and northwest of Wenatchee. Many of these showed characteristic evidence of type 2 weathering, that is D-lines on exposed members such as rails and curbs. Many of these structures also showed open cracks indicating the possibility of abnormal expansion. Aggregates which were used in a number of the bridges in this area have been selected for study in the cooperative work which has been started.

Typical type 2 D-line deterioration was found on a number of bridges in the vicinity of and to the southwest of Spokane. In some cases D-lines had progressed to the point of actual disintegration. As in other localities, the most seriously affected parts of the structures were the exposed sections above the deck line such as rails, curbs, and rail posts indicating that weathering is the primary cause of the trouble.

The concrete in one of the grade-separation structures in Spokane is of particular interest. Typical pattern cracking on one of the end posts is shown in figure 13. The cracks appear to go deeper than the fine D-line cracks so often noted. There was more of this type of cracking in Washington than in Oregon. The problem is whether this type of failure is purely the result of physical effects of weathering or whether, as Tremper's recent tests indicate, the resistance of the concrete has been weakened through some reaction involving the alkalies in the cement.

OBSERVATIONS IN CALIFORNIA

The inspections in California were made to observe the recent condition of structures in which alkali-aggregate reaction has taken place. These are located in the southern portion of the State. Structures in the high altitudes in which some type 2 weathering has been reported were not inspected. Several bridges over the Salinas River were examined and their present condition compared to that at the time of a previous inspection in 1937. Very little evidence of further action could be found, indicating that the chemical reaction which probably caused the initial expansion cracking has ceased. The cracking shown in figure 5, *B*, is typical of the vertical cracking noted on these structures. Other examples of this action may be seen in figure 14. In figure 14, *B*, the section of road surface back of the joint contains a reactive fine aggregate whereas in the section in the foreground the fine aggregate is nonreactive.

Deterioration resulting from cement alkali-aggregate reaction in California has been thoroughly discussed by Stanton and others and it is not necessary to go into detail in this report. However, attention should be called to the difference in conditions in southern California and in the northern States. In California, type 4 deterioration is not complicated by frost action. On the other hand, the concrete in the Mount Rainier area and in other sections of the north where there may be an alkali-aggregate reaction, is subject to many cycles of freezing and thawing each year. Cracks in the concrete due to expansion will obviously weaken the concrete and make it an easy prey to frost action. In this connection the combined effect of internal expansion and salt spray is seen in the serious disintegration of the Ventura sea wall, which is in very bad condition.

During the California inspection the party visited Parker Dam on the Colorado River and Copper Basin Dam, nearby. Both structures and also the pavement in the traffic circle at Fresno (*3, 5*) show evidence of alkali-aggregate reaction. An interesting summary of the present knowledge of cement alkali-aggregate reaction, is contained in an appendix to the report of Committee C–1 on Cement, A. S. T. M., which appeared in 1943 (*1*). The summary contains a fairly complete list of references to reports on this subject.

WEATHERING AS AFFECTED BY DIFFERENTIAL WATER CONTENT

It is a basic principle of concrete making that, given sound materials, durability is governed largely by the quality of the cementing ingredient—that is, the cement paste—and, further, that the quality of the paste is controlled largely by its relative water content (the water-cement ratio). This principle is as firmly established today as it ever was although the relatively new principle of air entrainment may seem at first thought to run counter to the idea that durability is a function of the density of the cement paste.

The influence of the water-cement ratio on durability is so well-recognized today that virtually all specifications for concrete to be exposed to severe weathering either require directly that the water-cement ratio not exceed 6.0 gallons per sack of cement, or specify requirements for minimum cement content and consistency in such a way as to insure that this value is not exceeded. A study of specifications governing construction of the various bridges inspected on this trip indicates that, in general, proportions were used which would insure

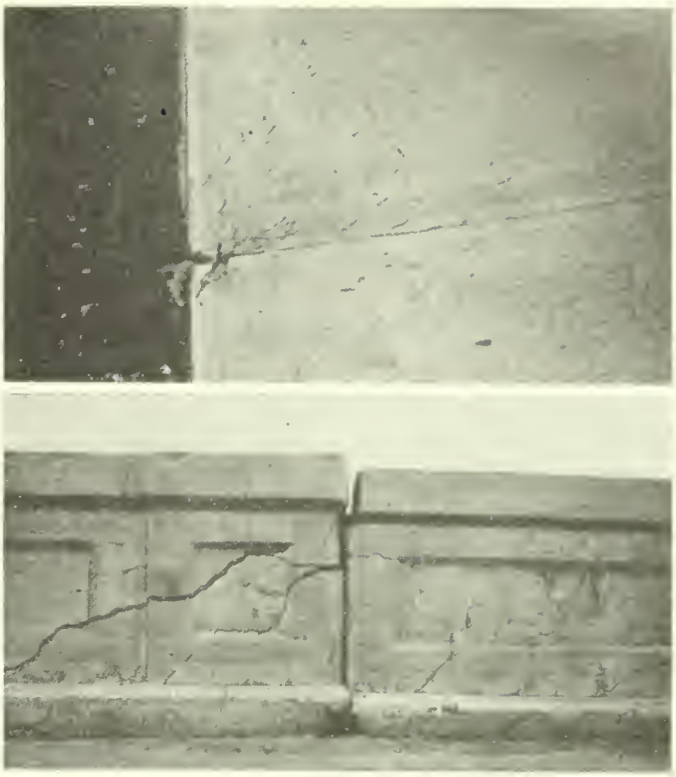

FIGURE 14.—TYPICAL TYPE 4 DETERIORATION CRACKING IN CALIFORNIA.

compliance with this requirement in combining materials at the mixer. Unfortunately the process of handling and depositing concrete in the forms, even when done under good supervision, frequently results in segregation, nonuniform compaction (particularly at joints), bleeding or water gain, and so forth. Ideally, concrete should be absolutely uniform in composition but variable factors all combine to make a product which actually may be far from uniform. The result is that even though the water-cement ratio of the mix as designed may be within the required limits, the actual water content in certain parts of the structure may be too high for adequate protection. The nonuniform distribution of concrete within a member as well as batch-to-batch variations in the quality of the concrete as mixed, variations in curing conditions, etc., all undoubtedly combine to produce the nonuniformity in resistance to weathering which is so often observed in various parts of the same structures. It is felt that at least some of the troubles which were noted during these inspections can be attributed to these conditions.

The writer would like to suggest adoption of a modification in placing procedure recently suggested to him by Mr. F. R. McMillan, of the Portland Cement Association. To prevent accumulation of a layer of relatively porous concrete of high water content in the top surface of vertical lifts, Mr. McMillan proposes the following: Instead of immediately striking off the concrete when the top of the form is reached, allow it to pile up 2 or 3 inches above the form, leaving it undisturbed for an hour or so in order that the excess water resulting from bleeding may accumulate in this layer. As soon as bleeding has ceased, strike off and finish in the usual way. The usual porous layer, instead of remaining in the concrete as a potential source

of weakness, will be wasted. The only loss will be a little concrete, the cost of which is a small price to pay for the elimination of a very common defect.

AIR ENTRAINMENT DISCUSSED

Application of the principle of air entrainment offers the most promising immediate solution to the problem of how to obtain more durable concrete. Originally suggested as a means for controlling salt scaling on pavements, it now appears that, in addition, resistance to freezing and thawing in general is greatly improved by the use of an air-entraining agent in the concrete. The effectiveness of air entrainment seems to lie in the fact that numerous minute air voids are incorporated in the concrete, and that these voids seem to act as cushions against the expansive force produced by freezing water. In any event, both laboratory tests and field experience indicate that substantial improvement in durability is obtained with little sacrifice in strength provided proper control is exercised in the use of the material.

At the present time air can be entrained in two ways; first, by the use of "Air-Entraining Portland Cement," a cement in which the air-entraining agent has been interground during manufacture and second, by adding the air-entraining agent directly to the concrete at the time of mixing. For the time being the writer favors the practice of adding the material at the mixer for several reasons. The amount of air that will be entrained in any particular case depends upon a number of factors in addition to the amount of active agent in the cement. The kind and grading of the aggregates, the type of mixer and the mixing time all affect the result, so that merely fixing the amount of air-entraining agent to be carried by the cement, or even fixing the amount of air which will be entrained in a standard test mortar as is done in the latest revision of A. S. T. M Specification C–175 is not sufficient. The only way in which the effect of all of the variables can be taken into account is to place control in the hands of the engineer on the job, permitting him to adjust the amount of air-entraining agent depending upon the particular conditions under which he is working.

It is quite important that close control be exercised because of the rather narrow limits of air content that must be maintained for optimum results. At the present time it seems that a total air content of from 3 to 5 percent based on the theoretical weight of the air-free concrete will be about right. Less than 3 percent may not be sufficient for maximum durability, whereas when the total air content exceeds 5 percent a substantial loss in strength without any further increase in durability may result. In a report published in June 1944 H. F. Gonnerman (*2*) of the Portland Cement Association presents the results of tests showing the effect of using both air-entraining portland cements and air-entraining materials added to the batch at the mixer. Several air-entraining agents were tested and if properly used should give satisfactory results. However, at the moment, the material supported by the greatest background of actual experience is Vinsol resin. This material has been used to a considerable extent interground with the cement. It has also been used, in the form of a Vinsol resin-sodium hydroxide solution, by adding to the batch at time of mixing. A weight test on the fresh concrete will indicate quite accurately the amount of this solution to add in any given case to produce the required air content. In addition to Vinsol resin, the material known commercially as Darex has also been approved for use in the manufacture of air-entraining portland cement, under A. S. T. M. Specification C–175. This material may also be added at the mixer as an air-entraining agent.

Although the writer favors the practice of adding the air-entraining agent at the mixer, it is realized that there are certain construction problems involved in this practice which are avoided by the use of air-entraining cement. The difficulty of properly controlling the addition on the job of very small quantities of such active materials as Vinsol resin is recognized. With Vinsol resin there is also the problem of preparing the material for use. To many engineers these practical matters outweigh the theoretical advantages of the procedure recommended in this report. However, it should be emphasized again that the really important matter is the amount of air which is entrained in the concrete as mixed on the job.

Not only must the air content be sufficiently high to insure the desired improvement in durability but it must not be so high as to jeopardize strength, bond-with-steel or any other essential property of the concrete. The optimum range appears to correspond to a total air content of from 3 to 5 percent. This is a rather narrow range and will require close control. Regardless of the method used to obtain air entrainment (by addition of an air-entraining agent at the mixer or the use of air-entraining cement), provision for such control during construction must be made in the specifications which govern the work. This can be done through the use in the field of a weight test such as A. S. T. M. C—138–44, with provision in the specifications for making such changes in materials, proportions, or methods of mixing as may be necessary to obtain the proper air content at all times.

There are certain indirect benefits associated with the use of air-entraining agents that should be mentioned. They have a distinctly plasticizing effect, especially in the leaner mixes, and, in general, reduce bleeding or water gain substantially. On this account it should be possible to obtain more uniform placement and thus avoid some of the segregation troubles to which reference has already been made. The additional plasticity makes it possible to reduce the sand content about 3 percent, thus at least partially compensating for the increased yield (lower cement content) which results from the air entrainment. This adjustment also tends to overcome the loss in strength which almost invariably accompanies increase in air content.

MODERN VERSUS OLD-FASHIONED CEMENT

Many of the older bridges covered by these inspections were in surprisingly good condition. In reviewing their condition with respect to age they seem to fall roughly into two groups; those built during the twenties and earlier and those built subsequent to 1930. It can be stated definitely that, on the whole, the bridges in the first group show less disintegration than those in the second group. Of 36 bridges built before 1930—24, or 67 percent, were free from defects other than those which could be classified as type 1 deterioration. On the other hand, of 101 bridges built after that date only 28, or 27 percent, were so classified. A similar comparison has been furnished by Mr. Paxson. He lists 61 structures on U S 30, between Umatilla and Ontario. Of the 37 built prior to 1930—29, or 78 percent, have

according to his record shown no evidence of weathering, whereas of the 24 built after that date only 9, or 37 percent, are so listed.

It is reasonable to suppose that, by and large, construction procedures are just as good now as they were 20 years ago. Presumably they are better. Likewise, engineering supervision has, on the average, improved. Certainly it has not deteriorated. The general character of the aggregates has not changed, although methods of processing, grading, and other factors tending to uniformity have improved. In view of these facts, it is not surprising that engineers should wonder about the cement. It is the one remaining factor. Is there some characteristic of modern cements not found in the old-fashioned type that tends to lower resistance to weathering?

The one outstanding difference is, of course, fineness. In 1917 the A. S. T. M. specifications required that not more than 22 percent be retained on the No. 200 sieve. The current specifications for type I portland cement requires a minimum specific surface of 1,600 square centimeters per gram, a limit which is roughly equivalent to 3 or 4 percent retained on the No. 200 sieve. The increase in fineness has come about gradually and largely as the result of demands on the part of users for high early strength. This demand culminated in 1930 in the adoption of specifications for high early strength cement, with still greater fineness.

Quite recently a very interesting theory has been advanced which may account for the apparent fact that modern cements, ground to a fineness corresponding to a specific surface of, say 1,800 square centimeters per gram do not make as durable concrete as the more coarsely ground cements in use 25 years ago. According to this theory, the maximum limit of 2 percent sulfur trioxide which has been carried for years in the A. S. T. M. specifications, while satisfactory for the coarser ground product, is not high enough for the more active, finer ground, modern cements. This applies particularly to cements high in tricalcium aluminate (C_3A) as it is largely for the purpose of regulating the hydration of this particular compound that the gypsum is employed. Recent investigations by the Portland Cement Association indicate that from the standpoint of both strength and drying shrinkage, cements of high and moderately high C_3A are improved by adding higher percentages of gypsum than are allowed by the present specifications. Furthermore, even cements of low C_3A content appear to be similarly improved if the alkali content is high. There are also some indications that drying shrinkage may be influenced by the alkali content of the cement independently of the C_3A content. The association is now engaged in an intensive study of this problem, with the idea of determining by means of laboratory freezing and thawing tests whether the use of larger quantities of gypsum will improve durability. If this is found to be the case, the sooner the specifications are changed the better.

NEED FOUND FOR RESTRICTION OF ALKALI CONTENT OF CEMENT FOR USE IN CERTAIN AREAS

This inspection has confirmed the belief that there is need for restrictions on the alkali content of portland cement when used in areas where alkali-reactive aggregates are found. It is true that all of the aggregates in these areas, may not be reactive. However, at the present time there are no tests, so far as the writer knows, that will reveal this characteristic in a reasonable length of time. The conventional expansion bar test requires several months at least. Furthermore, as pointed out by Tremper, cements high in alkali may react with certain aggregates in such a way as to weaken the resistance of the concrete to freezing and thawing even though the reaction is not sufficient to cause expansion in the bar test. For these reasons it seems wise, at least for the time being, to restrict the percentage of sodium and potassium oxide in the cement to 0.60 or less, calculated as Na_2O, in all areas where there is any evidence that reactive aggregates may be found.

This survey indicates that preventive steps should be taken in the area in southern California to which attention has been called by Stanton and others; in eastern Oregon and eastern Washington, including the area adjacent to Mount Rainier National Park; in the Yellowstone National Park, and possibly other areas in Wyoming; and in the area around Kimball, Nebr. Further information regarding the cement alkali-aggregate reaction will be available at the conclusion of the cooperative tests now under way.

SUMMARY OF RECOMMENDATIONS

For convenience the recommendations made in this report, insofar as they have application in the immediate revision of specifications, are summarized below:

That, where concrete is to be exposed to severe frost action, specifications be revised, where necessary, to require, in addition to a suitable cement content, that the free water content of the mix shall, in no case, exceed 6.0 gallons per sack of cement.

That, where concrete structures or portions of structures (including pavements) will be exposed to severe frost action, provision be made to entrain sufficient air in the fresh concrete so as to produce a total air content of from 3 to 5 percent, based on the theoretical weight of the air-free concrete.

That the desired air entrainment be obtained preferably by adding an approved air-entraining agent to the concrete at the time of mixing, in such quantity as will maintain the percentage of air within the limits specified.

That approval of any material proposed for use as an air-entraining agent be based on data obtained from either research or field use, or both, that are sufficiently comprehensive to demonstrate to the satisfaction of the contracting agency that the proposed material, when used as required, will not seriously affect the strength or other essential properties of the concrete.

That, when the desired air entrainment is to be obtained by the use of an air-entraining cement: (1) The cement meet the requirements for air entrainment given in A. S. T. M. Specification C 175–44 T; and (2) the specifications authorize the engineer to require such changes in materials, proportions, or methods of mixing as may be necessary from time to time to maintain the percentage of air within the limits specified.

That, in all areas where tests or previous experience indicate that alkali-reactive aggregates may be encountered, specifications for portland cement be modified by requiring that the total percentage of sodium oxide plus 0.658 times the percentage of potassium oxide shall not exceed 0.60.

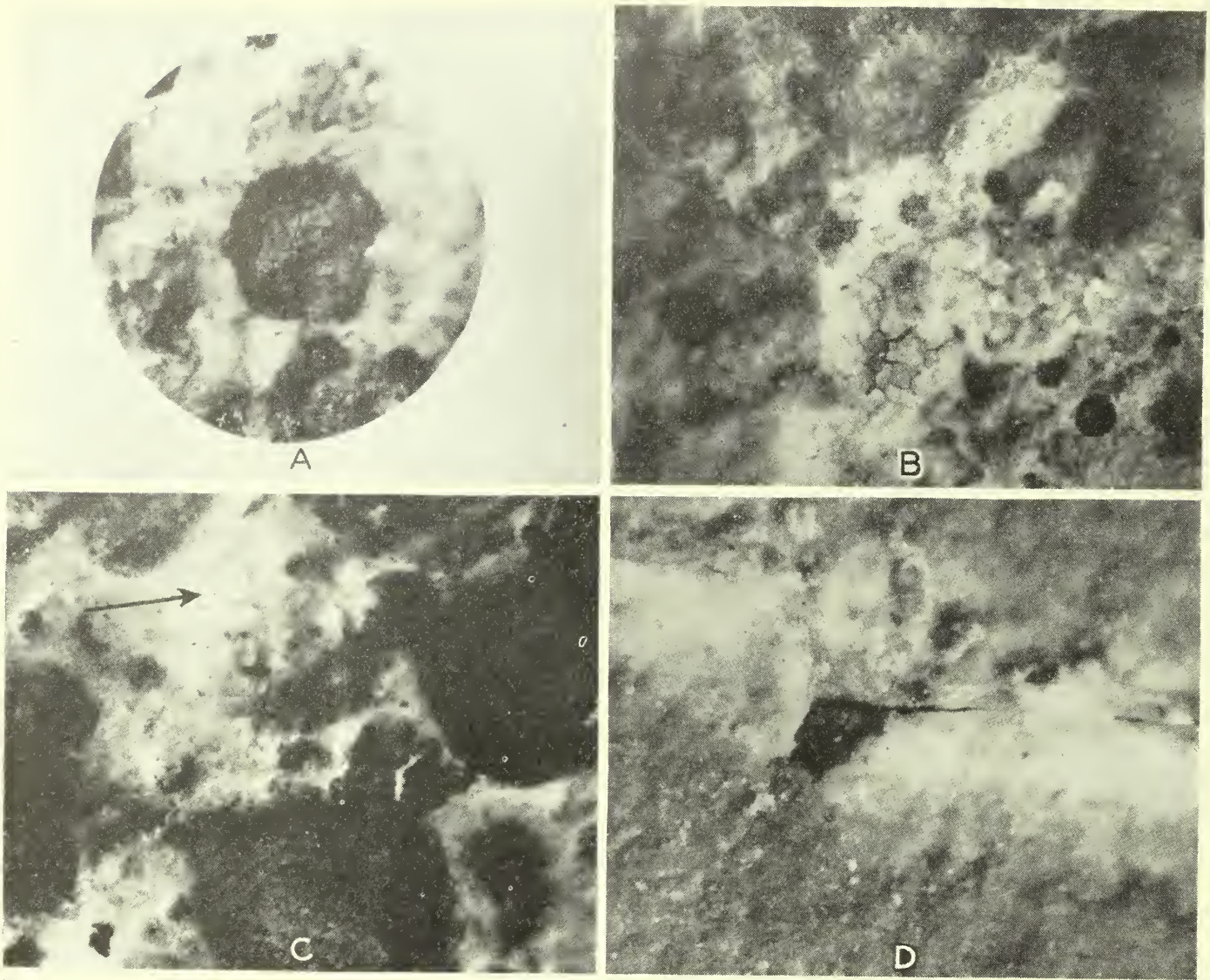

FIGURE 15.—(*A*) DEPOSIT OF GEL SHOWING SHRINKAGE CRACKS ON CLEAR FORK CREEK BRIDGE IN WASHINGTON, (*B*) THE FRACTURED SURFACE OF AUTOCLAVE BAR SHOWING A DEPOSIT OF GEL, (*C*) DEPOSIT OF GEL SHOWING SHRINKAGE CRACKS ON GIBBON RIVER BRIDGE IN YELLOWSTONE NATIONAL PARK, AND (*D*) GEL-FILLED CRACK ON SURFACE OF CONCRETE ON DEADWOOD CREEK BRIDGE IN WASHINGTON.

APPENDIX I.—REPORT OF PETROGRAPHIC LABORATORY ON SAMPLES OF CONCRETE OBTAINED DURING INSPECTION TRIP

During these inspections, a number of small samples of concrete were collected. In some instances samples were taken from portions of structures in which distress was very evident, while in other cases the samples were collected from apparently sound concrete. These samples were from the following bridges:

Gibbon River Bridge, Yellowstone Park.
North Santiam River Bridge, Oregon.
Marion Creek Bridge, Oregon.
Purcell Creek Bridge, Washington.
Clear Fork Creek Bridge, Washington.
Deadwood Creek Bridge, Washington.
Bridge in Tumwater Canyon, Washington.
Monterey Trestle, California.

Samples from the Marion Creek bridge in Oregon and the Purcell Creek bridge in Washington appeared sound in every respect. The concrete was homogeneous, dense, and as far as could be observed there were no indications of alkali reaction.

The sample of concrete from the Gibbon River bridge in Yellowstone National Park had an excessive number of voids throughout. Several pockets of dried gel-like material were observed. Calcium-sulphoaluminate crystals were also noted in some of the voids and aggregate sockets. A few of the aggregate particles exhibited rims of weathering which may or may not have been pre-concrete in origin. Figure 15, *A* shows a gel deposit in one of the voids.

Remains of gel formation, showing characteristic shrinkage cracks, were observed in the concrete taken from the Clear Fork Creek and Deadwood Creek bridges in Washington. The sample from the Clear Fork Creek bridge exhibited indications of alkali-aggregate reaction in the formation of a partially transparent exudation on some of the aggregate faces as well as in the voids. Figure 15, *B*, illustrates a deposit of gel formation in concrete from this structure.

Aside from the gel formations in voids and on aggregate faces, the concrete from the Deadwood Creek bridge contained dried gel in one of the radiating cracks on one face of the concrete that had been exposed to the elements. This gel had caused a slight expansion in the concrete adjoining the crack. A view of this is shown in figure 15, *C*.

The sample taken from the bridge in Tumwater Canyon, Washington, was characterized by lack of gel relics, but an excessive amount of calcium-sulphoaluminate needles was noted. Considerable carbonation was also observed in some of the aggregate sockets. These conditions indicate the possibility of freezing and thawing action as the reason for distress noted in the concrete.

Deposits of a white, semihard substance were observed in the concrete taken from the North Santiam River Bridge, Oregon. These deposits were found not only in the aggregate sockets but scattered throughout the matrix as well. This material proved

to be a carbonate and a gel-like substance. Considerable quantities of calcium-sulphoaluminate needles were also observed. These observations point strongly to distress caused by the action of freezing and thawing.

The concrete from the Monterey Trestle, California, although apparently sound in appearance, contained a few areas of opaline material. Some slight evidence of alkali-aggregate reaction was observed in a few areas showing the characteristic gel shrinkage cracks.

Figure 15, *D* illustrates a gel deposit found in a broken autoclave bar fabricated with a fine aggregate containing opal.

APPENDIX II.—METHODS USED BY THE OREGON STATE HIGHWAY DEPARTMENT FOR PATCHING AND WATERPROOFING DISINTEGRATED CONCRETE

The following is an outline of the procedure used by the Oregon State Highway Department in patching disintegrated concrete and waterproofing concrete surfaces for the prevention of disintegration. Considerable success has been attained in Oregon by following the methods as herein described.

REMOVAL AND PATCHING OF DISINTEGRATED CONCRETE

When disintegration of the commonly called "D-line" type has taken place, it usually affects the edges and corners of curbs handrails, wingwalls, etc. To prevent further progression of this type of disintegration, it is necessary to completely remove all disintegrated material, great care being taken to reach sound concrete beyond the extent of the "decayed" area. This can be accomplished by the use of hammers and chisels on small areas or by paving breakers or chipping hammers on larger areas. If the disintegration has reached an advanced stage, the complete removal and replacement of that portion of the structure may be necessary. The importance of the removal of all traces of disintegrated material cannot be overemphasized. Experience has shown that often the workman will remove all visible affected material, then place a patch on what was, in his judgment, sound concrete only later to discover that the material adjacent to the patch continues to disintegrate. A good rule seems to be to remove all material thought necessary, and then remove some more.

After the removal of the disintegrated concrete a patch is applied, the success of which depends upon, (*a*) securing a bond to the parent concrete, (*b*) overcoming the tendency of the patch to shrink after placement, and (*c*) proper curing. All places to be patched should be chipped out to secure not less than three-fourths-inch thickness for the patch. The edges of the patch should be square, that is, no feather edges. All surfaces should be clean and rough so as to secure a good bond and should be thoroughly saturated by several applications of water. The preshrinkage of mortar is required for all patches. This is done by mixing the mortar well ahead of use and letting it stand. The time required for preshrinkage of mortar varies with the different brands of cement and climatic conditions of temperature and humidity, and is best determined by experiment on the job. In general, the mortar thus preshrunk should be susceptible to use without the addition of more water before reworking it for application. After the patch has been applied, proper care in curing must be taken by keeping the patch covered with wet burlap 6 to 10 hours, after which it can be covered with damp sand or burlap until the concrete has thoroughly taken its set.

New concrete patches should be allowed one or two weeks to dry out before applying the waterproofing treatment. New concrete should be given a neutralizing wash to prevent saponification of the linseed oil used in the waterproofing treatment. A solution consisting of three pounds of zinc-sulphate crystals to a gallon of water is brushed over the surface to be treated and allowed to dry to 18 hours. When thoroughly dry, all crystals on the surface are removed by wire brushing. This treatment is not necessary on old concrete.

WATERPROOFING TREATMENT

Cleaning.— Before the waterproofing treatment is applied, it is necessary that the concrete surface be clean and dry. To clean the surface, brush with a wire bristle brush removing all dust and loose material. Road oil or grease can be removed by scrubbing with gasoline or a solvent. Efflorescence can be removed by scrubbing with a 10-percent solution of hydrochloric acid. When water is used in cleaning, ample time must be allowed to permit thorough drying of the concrete surfaces before applying the waterproofing.

Application of Linseed Oil.— After the surface has been properly prepared and is clean and dry, two coats of hot linseed oil are to be applied.

The first coat consists of a mixture of 50 percent raw linseed oil and 50 percent turpentine heated to a temperature of 175° F. and applied with an ordinary paint brush. Better results will be obtained if atmospheric temperature is above 65°. Allow the first coat to set 24 hours before applying second coat.

After the first coat of the linseed oil-turpentine mixture has set, spots will be evident where the concrete is more porous than the remainder of the surface treated. These spots should be spot treated with the hot mixture and allowed to set before the second coat of linseed oil is applied.

The second coat shall consist of undiluted raw linseed oil heated to 175° and applied in the same manner as the first coat. When this coat is thoroughly dry, the surface can be painted with an approved concrete paint.

CONCRETE PAINT

After the waterproofing treatment has been applied, the entire treated surface should be given two coats of any standard outside white lead and oil paint tinted to the desired shade. A concrete color can be obtained by the addition of lampblack and raw sienna ground in oil. The Oregon standard white paint formula is as follows:

	Percent
Paint composition:	
Pigment not less than	70
Vehicle not more than	30
Pigment composition:	
White lead carbonate	40.0 to 45.0
Titanium barium pigment	35.0 to 40.0
Zinc oxide	15.0 to 20.0
Tinting pigments, if required	0.0 to 5.0

The first coat should be thinned by the addition of 2 quarts of turpentine and 2 quarts of boiled linseed oil to the gallon of the above formula. The second coat should be thinned with about 1 quart of boiled linseed oil to the gallon of paint so as not to get a heavy pigment coat that will be susceptible to scaling, but which must be heavy enough to brush out uniformly and evenly.

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THE FEDERAL-AID HIGHWAY ACT OF 1944

[Public Law 521—78th Congress]

AN ACT

To amend and supplement the Federal-Aid Road Act, approved July 11, 1916 as amended and supplemented, to authorize appropriations for the post-war construction of highways and bridges, to eliminate hazards at railroad-grade crossings, to provide for the immediate preparation of plans, and for other purposes.

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That, when used in this Act, unless the context indicates otherwise—

The term "construction" means the supervising, inspecting, actual building, and all expenses incidental to the construction or reconstruction of a highway, including locating, surveying, and mapping, costs of rights-of-way, and elimination of hazards of railway-grade crossings.

The term "urban area" means an area including and adjacent to a municipality or other urban place, of five thousand or more, the population of such included municipality or other urban place to be determined by the latest available Federal census. The boundaries of urban areas, as defined herein, will be fixed by the State highway department of each State subject to the approval of the Public Roads Administration.

The term "rural areas" means all areas of the State not included in "urban areas".

The term "secondary and feeder roads" means roads in rural areas, including farm-to-market roads, rural-mail routes, and school-bus routes, and not on the Federal-aid system.

Sec. 2. For the purpose of carrying out the provisions of the Federal Highway Act, approved November, 9, 1921, as amended and supplemented, there is hereby authorized to be appropriated the sum of $1,500,000,000 to become available at the rate of $500,000,000 a year for each of three successive post-war fiscal years: *Provided,* That of the sums authorized to be appropriated for the first of such fiscal years $100,000,000 may be appropriated in accordance with the provisions of this Act to become available immediately upon apportionment of the authorization for said fiscal year for the making of surveys and plans and for construction: *Provided further,* That except for the sum appropriated pursuant to the preceding proviso, no part of the funds made available pursuant to this Act shall be used to pay costs incurred under any construction contract entered into by any State before the beginning of the first post-war fiscal year. The first post-war fiscal year shall be that fiscal year which ends on June 30th following the date proclaimed by the President as the termination of the existing war emergency, or following the date specified in a concurrent resolution of the two Houses of Congress as the date of such termination, or following the date on which the Congress by a concurrent resolution of the two Houses finds as a fact that the war emergency hereinbefore referred to has been relieved to an extent that will justify proceeding with the highway construction program provided for by this Act, whichever date is the earliest. The authorization for the first post-war fiscal year shall be apportioned among the States within thirty days from the passage of this Act. The authorization for the second post-war fiscal year shall be apportioned among the States within twelve months after the date of such termination or finding as above specified, and the authorization for the third post-war fiscal year shall be apportioned among the States within twelve months after the apportionment of the authorization for the second post-war fiscal year. As soon as the funds for each of the post-war fiscal years have been apportioned, the Commissioner of Public Roads is authorized to enter into agreements with the State highway departments for the making of surveys and plans, the acquisition of rights-of-way, and the post-war construction of projects. His approval of any such agreement shall be a contractual obligation of the Federal Government for the payment of its pro rata share of the cost of construction: *Provided, however,* That the Commissioner of Public Roads shall not, as a condition of approval of any project for Federal aid hereunder, require any State to acquire title to, or control of, any marginal land along the proposed highway in addition to that reasonably necessary for road surfaces, median strips, gutters, ditches, and side slopes and sufficient width to provide service roads for adjacent property to permit safe access at controlled locations in order to expedite traffic, promote safety, and minimize roadside parking.

Sec. 3. The sum authorized in section 2 for each year shall be available for expenditures as follows:

(a) $225,000,000 for projects on the Federal-aid highway system.

(b) $150,000,000 for projects on the principal secondary and feeder roads, including farm-to-market roads, rural free delivery mail and public-school bus routes, either outside of municipalities or inside of municipalities of less than five thousand population: *Provided,* That these funds shall be expended on a system of such roads selected by the State highway departments in cooperation with the county supervisors, county commissioners, or other appropriate local road officials and the Commissioner of Public Roads: *Provided further,* That in any State having a population density of more than two hundred per square mile, as shown by the latest available Federal census, the said system may be selected by the State highway department with the approval of the Commissioner of Public Roads without regard to included municipal boundaries: *Provided further,* That any of such funds for secondary and feeder roads which are apportioned to a State in which all public roads and highways are under the control and supervision of the State highway department may, if the State highway department and the Commissioner of Public Roads jointly agree that such funds are not needed for secondary and feeder roads, be expended for projects in such State on the Federal-aid highway system.

(c) $125,000,000 for projects on the Federal-aid highway system in urban areas.

Sec. 4. After making the deductions for administration, research, and investigations as provided in section 21 of the Federal Highway Act of 1921, the sums authorized shall be apportioned as follows:

(a) The $225,000,000 per year available for projects on the Federal-aid highway system shall be apportioned among the States as provided in section 21 of the Federal Highway Act.

(b) The $150,000,000 per year available for projects on the secondary and feeder roads shall be apportioned among the States in the following manner: One-third in the ratio which the area of each State bears to the total area of all the States; one-third in the ratio which

the rural population of each State bears to the total rural population of all the States, as shown by the Federal census of 1940; and one-third in the ratio which the mileage of rural delivery and star routes in each State bears to the total mileage of rural delivery and star routes in all the States: *Provided*, That no State shall receive less than one-half of one per centum of each year's allotment under subsection (a) and this subsection.

(c) The $125,000,000 per year available for projects on highways in urban areas shall be apportioned among the States in the ratio which the population in municipalities and other urban places, of five thousand or more, in each State bears to the total population in municipalities and other urban places, of five thousand or more, in all the States as shown by the latest available Federal census: *Provided*, That Connecticut and Vermont towns shall be considered municipalities regardless of their incorporated status.

(d) Any sums apportioned to any State under the provisions of this section shall be available for expenditure in that State for one year after the close of the fiscal year for which such sums are authorized, and any amount so apportioned remaining unexpended at the end of such period shall lapse: *Provided*, That such funds shall be deemed to have been expended if covered by formal agreement with the Commissioner of Public Roads for the improvement of a specific project as provided by this Act.

SEC. 5. (a) The Federal share payable on account of any project provided for by the funds made available under the foregoing provisions of this Act shall not exceed 50 per centum of the construction cost thereof other than costs of rights-of-way, and as to costs of rights-of-way shall not exceed one-third of such costs: *Provided*, That in the case of any State containing unappropriated and unreserved public lands and nontaxable Indian lands, individual and tribal, exceeding 5 per centum of the total area of all lands therein the Federal share shall be increased in each of the three post-war years by a percentage of the remaining cost equal to the percentage that the area of all such lands in such State is of its total area: *Provided further*, That the entire construction cost of projects for the elimination of hazards of railway-highway crossings, including the separation or protection of grades at crossings, the reconstruction of existing railroad grade crossing structures, and the relocation of highways to eliminate grade crossings, may be paid from Federal funds, except that not more than 50 per centum of the right-of-way and property damage costs, paid from public funds, on any such project, may be paid from Federal funds: *Provided further*, That not more than 10 per centum of the sums apportioned to any State under the terms of this Act for each of such post-war fiscal years shall be used for such railway-highway projects, to be expended in accordance with the Federal Highway Act, as amended and supplemented, and the provisions of this section.

(b) Any railway involved in any project for the elimination of hazards of railway-highway crossings paid for in whole or in part from funds made available under this Act, shall be liable to the United States for a sum bearing the same ratio to the net benefit received by such railway from such project that the Federal funds expended on such project bear to the total cost of such project. For the purposes of this subsection, the net benefit received by a railway from any such project shall be deemed to be the amount by which the reasonable value of the total benefits received by it from such project exceeds the amount paid by it (including the reasonable value of any property rights contributed by it) toward the cost of such project; and in no case shall the total benefits to any railway or railways be deemed to have a reasonable value in excess of 10 per centum of the cost of any such project. The liability of any railway to the United States with respect to any such project may be discharged by paying to the United States, within six months after the completion of such project, such amount as the Commissioner of Public Roads determines to be the amount of such liability. Any such determination of the Commissioner shall be made on the basis of recommendations made to him by the State highway department and on the basis of such other information and investigation, if any, as the Commissioner deems necessary or proper. If any such railway has failed so to discharge its liability to the United States with respect to any project within six months after the completion thereof, the Commissioner of Public Roads shall request the Attorney General to institute proceedings against such railroad for the recovery of the amount for which it is liable under this subsection. The Attorney General is authorized to bring such proceedings on behalf of the United States in the appropriate district court of the United States, and the United States shall be entitled in such proceedings to recover such sums as it is considered and adjudged by the court that such railway is liable for in the premises. Any amounts paid to or recovered by the United States under this subsection shall be covered into the Treasury as miscellaneous receipts.

SEC. 6. If the Commissioner of Public Roads shall determine that it is necessary for the expeditious completion of projects undertaken pursuant to this Act, he may advance to any State from funds heretofore or hereafter made available the Federal share of the cost thereof to enable the State highway department to make prompt payments for work as it progresses: *Provided*, That such State, after June 30, 1945, does not divert to other than highway uses road user revenues in violation of section 12 of the Highway Act of June 18, 1934. The funds so advanced shall be deposited in a special trust account by the State treasurer, or other State official authorized under the laws of the State to receive Federal-aid highway funds, to be disbursed solely upon vouchers approved by the State highway department for work actually performed in accordance with plans, specifications, and estimates approved by the Public Roads Administration under the provisions of this Act. Any unexpended balances of funds so advanced shall be returned to the credit of the appropriation from which the funds have been advanced: *Provided*, That any advance made to any State under the provisions of this section and not repaid shall be deducted from any apportionment allocated to such State under the provisions of this Act for the year next succeeding the year in which such advance is made, and no agreement made in accordance with the provisions of section 2 of this Act shall be valid for any pro rata share of the cost of construction in excess of such apportionment less such advance.

SEC. 7. There shall be designated within the continental United States a National System of Interstate Highways not exceeding forty thousand miles in total

extent so located as to connect by routes, as direct as practicable, the principal metropolitan areas, cities, and industrial centers, to serve the national defense, and to connect at suitable border points with routes of continental importance in the Dominion of Canada and the Republic of Mexico. The routes of the National System of Interstate Highways shall be selected by joint action of the State highway departments of each State and the adjoining States, as provided by the Federal Highway Act of November 9, 1921, for the selection of the Federal-aid system. All highways or routes included in the National System of Interstate Highways as finally approved, if not already included in the Federal-aid highway system, shall be added to said system without regard to any mileage limitation.

SEC. 8. With the approval of the Federal Works Administrator, not to exceed 1½ per centum of the amount apportioned for any year to any State under the Federal Highway Act, as amended and supplemented, except sections 3 and 23 thereof, shall hereafter be used with or without State funds for surveys, plans, engineering, and economic investigations of projects for future construction in such State, on the Federal-aid highway system and extensions thereof within municipalities, on secondary or feeder roads, urban highways or grade-crossing eliminations, and for highway research necessary in connection therewith.

SEC. 9. For the purpose of carrying out the provisions of section 23 of the Federal Highway Act (42 Stat. 218), as amended and supplemented, there is hereby authorized to be appropriated (1) for forest highways the sum of $25,000,000 for the first post-war fiscal year and a like amount for each of the second and third post-war fiscal years; and (2) for forest development roads and trails the sum of $12,500,000 for the first post-war fiscal year and a like amount for each of the second and third post-war fiscal years: *Provided*, That the apportionment for forest highways in Alaska shall be for each year $1,500,000 and that such additional amount as otherwise would have been apportioned to Alaska for each of said years shall be apportioned among those States, including Puerto Rico, whose forest highway apportionment for such year otherwise would be less than 1 per centum of the entire apportionment for forest highways for that year.

SEC. 10. (a) For the construction, reconstruction, improvement, and maintenance of roads and trails, inclusive of necessary bridges, in national parks, monuments, and other areas administered by the National Park Service, including areas authorized to be established as national parks and monuments, and national park and monument approach roads authorized by the Act of January 31, 1931 (46 Stat. 1053), as amended, there is hereby authorized to be appropriated the sum of $12,750,000, to become available at the rate of $4,250,000 a year for each of the three successive post-war fiscal years.

(b) For the construction and maintenance of parkways, to give access to national parks and national monuments, or to become connecting sections of a national parkway plan, over lands to which title has been transferred to the United States by the States or by private individuals, there is hereby authorized to be appropriated the sum of $30,000,000, to become available at the rate of $10,000,000 a year for each of the three successive post-war fiscal years.

(c) For the construction, improvement, and maintenance of Indian reservation roads and bridges and roads and bridges to provide access to Indian reservations and Indian lands under the provisions of the Act approved May 26, 1928 (45 Stat. 750), there is hereby authorized to be appropriated the sum of $6,000,000 for the first post-war fiscal year and a like amount for each of the second and third post-war fiscal years: *Provided*, That the location, type, and design of all roads and bridges constructed shall be approved by the Public Roads Administration before any expenditures are made thereon, and all such construction shall be under the general supervision of the Public Roads Administration.

SEC. 11. Federal highway funds shall not be used for the reconstruction or relocation of any highway giving access to an airport (if such airport has been constructed or extended after the date of enactment of this Act), or for the reconstruction or relocation of any highway which has been or may be closed or the usefulness of which has been or may be impaired by the location or construction of any airport (if such airport has been constructed or extended after the date of enactment of this Act), unless, prior to such extension or construction, as the case may be, the State highway department and the Public Roads Administration have concurred with the officials in charge of the airport that the location of such airport or extension thereof and the consequent reconstruction or relocation of the highway are in the public interest.

SEC. 12. On any highway or street hereafter constructed with Federal aid in any State, the location, form, and character of informational, regulatory, and warning signs, curb and pavement or other markings, and traffic signals installed or placed by any public authority, or other agency, shall be subject to the approval of the State highway department with the concurrence of the Public Roads Administration; and the Commissioner of Public Roads is hereby directed to concur only in such installations as will promote the safe and efficient utilization of the highways.

SEC. 13. If any section, subsection, or other provision of this Act or the application thereof to any person or circumstance is held invalid, the remainder of this Act and the application of such section, subsection, or other provision to other persons or circumstances shall not be affected thereby.

SEC. 14. This Act may be cited as the "Federal-Aid Highway Act of 1944".

Approved December 20, 1944.

U. S. GOVERNMENT PRINTING OFFICE: 1945

A TEST FOR DETERMINING THE EFFECT OF WATER ON BITUMINOUS MIXTURES

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by J. T. PAULS, Principal Highway Engineer, and H. M. REX, Materials Engineer

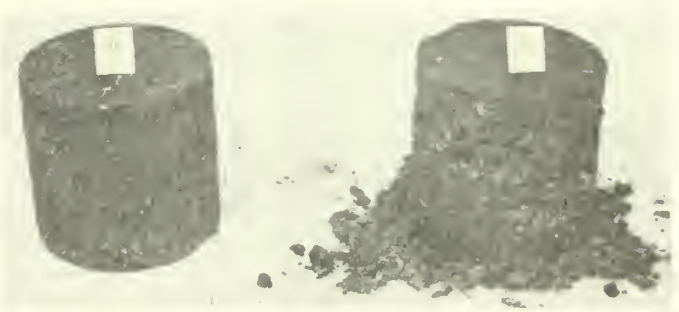
IT IS WELL KNOWN that bituminous mixtures containing aggregates of certain types and characteristics are susceptible to the action of water and, as a result, many tests and specifications have been developed to regulate, or to eliminate entirely, the use of the more water-sensitive materials.

One of the most familiar water-sensitive materials is clay. Specifications governing many types of bituminous road construction require that the aggregate shall be free from clay, or that the aggregate particles shall be free from adhering films of clay. Other requirements for control of this material limit the plasticity index and liquid limit for the aggregate fraction passing a No. 40 sieve. Volume change is one manifestation of the presence of clay in a bituminous mixture, and a swell test has been developed and widely used as a means of control.

A different type of water sensitivity has been found to be inherent in the aggregate itself. This characteristic has been described as the preferential affinity of aggregate particles for bitumen or water in the presence of both. An outward manifestation of this characteristic is the propensity of the aggregate particles to retain or lose a bituminous film when exposed to the action of water. An aggregate that tends to retain a large percentage of bituminous coating under such conditions has come to be classed as hydrophobic and one that tends to lose a bituminous film as hydrophilic.

In order to test the hydrophilic or hydrophobic properties of aggregates, several tests have been devised. A bibliography on the subject appears at the end of this report. The Nicholson test, the Oberbach test and modifications thereof, various scrubbing tests, etc., have been widely used by investigators. Whatever the advantage peculiar to each of these stripping tests, there are several serious disadvantages common to all. First, only one size fraction of the whole aggregate is usually selected for test. Second, estimation of the extent of stripping is made by visual observation—a method seriously lacking in precision. Third, results of such tests give no indication of the actual degree of adhesion of the bituminous film to the aggregate particle. When the less viscous bituminous materials are used with certain aggregates, it has been observed that water will loosen or separate the film from the aggregate particle without, however, disturbing the continuity of the film. Failure of this type is not reflected in the results of the usual stripping test. Fourth, and most significant, there is no direct relation between the results obtained by any of the stripping tests and the effects that will be obtained when the complete bituminous mixture containing the aggregate in question is exposed to the action of water.

It is believed that there is a distinct need for a test that will measure positively the effect of the presence of water-sensitive materials of any type in bituminous mixtures when such mixtures are exposed to the action of water. This report relates the development and describes the procedure of a test capable of performing this function, and presents data to demonstrate its usefulness.



EFFECT OF IMMERSING MOLDED SPECIMENS IN WATER. SPECIMEN A CONTAINS HYDROPHOBIC AGGREGATE AND SPECIMEN B CONTAINS HYDROPHILIC AGGREGATE. SPECIMEN B HAS LOST ALL STABILITY.

DISAPPOINTING RESULTS OF COLORADO EXPERIMENTAL SECTIONS LEAD TO LABORATORY STUDY

In the spring of 1942, a laboratory study was made of aggregates and bituminous materials that had been used in experimental road surfacing by the Colorado State Highway Department, in cooperation with the Public Roads Administration. It was intended to study the usefulness of a requirement for viscosity index in specifications for slow-curing (SC) liquid asphaltic materials by constructing and observing a number of test sections in which the only variable would be the viscosity index of the bituminous materials.

The experimental sections were constructed in the fall of 1941, and the work consisted of reconditioning and priming the base and mixing and laying a 1½-inch bituminous mat composed of SC–3 (slow-curing) material and local aggregates. Thirteen different asphaltic materials of SC–3 grade were used in the experiment, each section of which was 3,700 feet in length.

The aggregate used in the bituminous mixture was obtained from a local pit adjoining the highway. The pit material was fairly uniform in quality and grading and, judged by existing standards and specifications, gave no indication that unsatisfactory behavior would result from its use.

A typical grading is given below. Other physical characteristics of the aggregate used in the construction are shown in table 1.

TABLE 1.—*Physical properties of aggregate*

Aggregate	Specific gravity		Absorption	Voids by vibration
	Bulk	Apparent		
			Percent	*Percent*
17 percent retained on No. 4 sieve	2.70	2.62	1.76	-
83 percent passing No. 4 sieve	2.41	2.60	2.48	-
100 percent total aggregate...		2.60		28.4

Sieve size:	Total percent passing
¾-inch	100
⅜-inch	90
No. 10	70
No. 40	36
No. 200	15

The governing specifications for the bituminous material were as follows:

Water and sediment, percent	2−
Flash point, °F	200+
Furol viscosity, 140° F., seconds	150–300
Distillation test, total distillate off at—	
600° F., percent by volume	10−
680° F., percent by volume	20−
Tests on the distillation residue:	
Float test, 122° F., seconds	25+
Solubility in CCl_4, percent	99.5+
Oliensis test	Negative
Viscosity index	Variable [1]
Residue of 80 penetration, percent	65–75
Ductility tests on residue of 80 penetration:	
77° F., 5 cm. per min., cm	100+
39.2° F., ¼ cm. per min., cm	5+

Behavior of the test sections, however, was disappointing. Rapid deterioration of the surfacing began with the first rains. The surface softened to a slight depth after each rain and became slimy under traffic. Upon drying, the surface raveled, leaving the larger aggregate particles exposed. This action continued, with the result that by early spring in 1942 a considerable thickness of surface had been lost. Consequently, the objective sought in these experiments, namely, to determine whether or not the viscosity index is a significant test requirement, was not attained. Any differences attributable to variations in bituminous materials that might otherwise have been apparent were completely obscured by the predominant effect of the particular aggregate used.

LABORATORY STUDY SUGGESTS NEED FOR A MORE INFORMATIVE TEST

Samples of the aggregate and of the road mixtures had been sent to the laboratory at the time the road sections were built. The laboratory testing schedules for the aggregate included determinations of absorption, specific gravity, vibratory density, and stripping by the modified Nicholson test procedure.

In the Nicholson test, 95 grams of the aggregate passing a ⅜-inch sieve and retained on a No. 4 sieve is combined with 5 grams of bituminous material and mixed by hand until a uniform coating of the aggregate is obtained. This sample is then placed in an oven maintained at a constant temperature of 140° F. for 24 hours. After removing from the oven, the sample is allowed to cool to room temperature, after which approximately 50 grams is placed in an Erlenmeyer flask of 250 milliliters capacity, and covered with 175 milliliters of distilled water. The flask is then stoppered and placed in the rotating frame of the agitating machine. The speed of rotation is 43 rotations per minute. The water bath in the machine is maintained at 77° F. for the first 30 minutes of the test. At the expiration of 1, 3, 5, 10, 15, and 30 minutes from the time of starting the test, the machine is stopped and the area of aggregate coated with bitumen is estimated. The temperature of the water in the machine is then raised to 100° F., and agitation resumed for an additional 15 minutes, after which the machine is stopped and the extent of uncoating estimated as before. A final 15 minutes of agitation with the water bath at a temperature of 120° F. ends the test. It is believed that the last 15 minutes of agitation with the water at 120° F. is too severe to be significant except when hot mixtures are being tested. Therefore, only the results at 77° and 100° F. for time intervals up to 45 minutes are included in this report.

Stripping tests, by this method, were made on four samples prepared in the laboratory. The bituminous materials used were: (1) An SC–3 slow-curing material actually used in one of the test sections, and (2) an MC–3 cut-back material, characteristics of which are shown in table 7. The latter material was included so as to compare the results with a cut-back material. The aggregates were that used on the Colorado job and Potomac River gravel. The Potomac River gravel was tested in combination with the two grades of bituminous material for the purpose of comparison.

Results of these tests are given in table 2.

[1] Materials were selected to cover a wide range of viscosity index.

TABLE 2.—*Results of modified Nicholson tests on Colorado and Potomac River aggregates*

Source of aggregate	Type of bituminous material	Estimated area coated	
		Water at 77° F. (30 min.)	Water at 100° F. (45 min.)
		Percent	*Percent*
Potomac River gravel	SC–3	95	95
	MC–3	100	100
Colorado aggregate	SC–3	50	40
	MC–3	50	40

TABLE 3.—*Degradation of the Colorado aggregate by the action of water*

Sieve size	Weight		Loss	
	Original	After 24 hours in water		
	Grams	*Grams*	*Grams*	*Percent*
¾-inch–⅜-inch	78.0	73.1	4.9	6.7
⅜-inch–No. 4	100.0	87.2	12.8	12.8
No. 4–No. 8	100.0	89.8	10.2	10.2
No. 8–No. 16	100.0	89.9	10.1	10.1
No. 16–No. 30	100.0	87.7	12.3	12.3
No. 30–No. 50	100.0	85.9	14.1	14.1

TABLE 4.—*Results of laboratory tests on 3- by 3-inch cylinders molded from field mixtures*

Field mix section No.	Vertical swell after immersion in water, unconfined		Compressive strength				
			Strength of dry specimens	After immersion in water			
				1 day		7 days	
	1 day	7 days		Strength	Strength retained	Strength	Strength retained
	Percent	*Percent*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*	*Lb. per sq. in.*	*Percent*
1	1.4	4.9	43	19	44	5	12
2	2.4	4.0	46	7	15	4	9
3	2.5	3.8	59	6	10	4	7
4	1.5	2.9	52	17	33	5	10
5	.7	2.6	55	24	44	7	13
6	.8	2.2	38	21	55	7	18
7	1.2	2.9	54	17	31	4	7
8	1.0	2.5	38	17	45	7	18
9	.6	2.5	53	26	49	6	11
10	1.0	2.9	61	24	39	6	10
11	.7	2.7	64	33	52	10	16
12	.5	2.7	71	32	45	9	13
13	.7	2.5	69	33	48	8	12

TABLE 5.—*Results of immersion and compression tests on 3- by 3-inch cylinders composed of laboratory mixtures containing Colorado and Potomac River aggregates*

Aggregate	Bitumen		Vertical swell of unconfined specimens after immersion in water		Compressive strength			Retained strength after immersion	
	Grade and type	Proportion by weight of aggregate	1 day	7 days	As molded	After immersion in water at 77° F.			
						1 day	7 days	1 day	7 days
		Percent	*Percent*	*Percent*	*Lb. per sq. in*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*	*Percent*
Colorado pit run	SC–3 [1]	5.5	5.0	5.4	41	1	[2]	2	0
Colorado pit run	MC–3	6.0	3.5	4.4	79	4	3	5	4
Potomac River sand-gravel with limestone dust.[3]	SC–3 [1]	5.5	.5	.6	15	15	13	100	87
Potomac River sand-gravel with limestone dust.[3]	MC–3	6.0	.2	.2	31	32	30	103	97

[1] SC–3 material used in the field construction and in the stripping tests reported in table 2, p. 116.
[2] Disintegrated. [3] Size and grading made identical to that of the Colorado aggregate.

The Colorado aggregate showed substantially greater stripping tendencies than did the Potomac River gravel. It is also of interest to note that substitution of MC–3 for SC–3 material gave essentially identical results with both aggregates. However, the results of this single test on one fraction of the Colorado aggregate did not afford sufficient evidence to warrant a prediction of failure of the type that occurred on the test sections. It seemed evident that characteristics of the aggregate other than stripping had an important effect on the behavior of the bituminous mixture in its resistance to the action of moisture.

Early in the laboratory investigation it became apparent that degradation of the Colorado aggregate followed exposure to water. In order to establish some quantitative measure of this effect, the following soundness test was made.

Portions of six different size fractions were selected for test. After sieving to refusal, the several fractions, each resting on the respective sieve on which it was retained, were placed in pans of suitable depth. Distilled water was added to each pan to an elevation of about one-half inch above the aggregate. The pans and contents were covered. After 24 hours the sieves were removed from the water and the retained material dried, again sieved to refusal, and weighed. Results of this test are shown in table 3. The differences in weight shown represent the losses of the original fractions due to the action of water.

Such a break-down can occur only in a structurally weak aggregate. It is also reasonable to suppose that this tendency to degrade affected the results of the Nicholson test. No doubt considerable area reported as stripped represented newly exposed faces produced by splitting of the aggregate particles in water as the test proceeded.

Although results of the Nicholson test and the soundness test showed that the Colorado aggregate was strongly susceptible to water action, there was still lacking conclusive evidence by any laboratory test of how a bituminous mixture containing this aggregate would perform when exposed to the action of water. It was at this point that the idea was conceived of comparing the compressive strength of dry molded specimens of the mixtures with the strength of specimens that had been immersed in water.

Samples of the oil-aggregate mixture of each of the 13 sections, upon completion of the mixing operations in the field, had been sent to the laboratory. Using these mixtures, cylindrical specimens, 3 inches in diameter and 3 inches in height, were molded at room temperature using a molding pressure of 1,000 pounds per square inch, and maintaining the load for 1 minute.

Specimens from each mix were tested in compression immediately after molding, and after immersion in water for periods of 1 and 7 days. The volume change, or swell, of the immersed specimens, as indicated by the change in the vertical dimension, was also noted. The results of these tests are given in table 4.

In addition, mixtures composed of the Colorado aggregate as received and an SC–3 material were prepared in the laboratory. Mixtures containing Colorado aggregate and MC–3 material were also prepared. For the purpose of comparison similar mixtures were made in which each of the two bituminous materials was combined with Potomac River sand and gravel after it had been sieved and recombined to conform to the grading of the Colorado aggregate. Limestone dust was added to the Potomac River sand and gravel to bring the percentage of filler up to that naturally contained in the Colorado aggregate.

These laboratory mixtures were prepared by hand with the aggregates at air temperature and the bituminous materials heated to 175° F. Mixing was continued as long as improvement in coating could be observed. Prior to molding, the mixtures were placed in open pans in an oven, maintained at a temperature of 140° F. and cured for 24 hours. Cylindrical specimens 3 inches in diameter and 3 inches in height were molded from these mixtures, using a molding pressure of 3,000 pounds per square inch, maintained for 1 minute.

The test procedure for the laboratory-prepared mixtures was the same as that described for the field mixtures. One set of specimens was tested immediately after molding, one set after 1 day immersion in water, and one set after 7 days of water immersion.

Typical results of immersion and compression tests of the mixtures prepared in the laboratory are shown in table 5.

USEFULNESS OF THE IMMERSION-COMPRESSION TEST INDICATED

Results of immersion-compression tests on specimens of field and laboratory mixtures containing the Colorado aggregate show striking loss of stability after immersion in water. The higher stability retained by specimens molded from the field samples, compared to that of specimens made from mixtures prepared in the laboratory was undoubtedly due to the more extensive curing to which the field mixtures had been subjected in the process of road mixing and in the interval between mixing and testing. In the laboratory mixtures con-

taining Colorado aggregate, substitution of MC–3 material for SC–3 material resulted in very little improvement in retention of stability. Comparable mixtures consisting of Potomac River sand-gravel and limestone dust, with either grade of bituminous material, lost little strength even after 7 days immersion, and in these mixtures considerable improvement resulted from the substitution of MC–3 for SC–3 material.

In general the vertical swell of the unconfined specimens varied consistently with the loss of stability. It is believed however that the increase in vertical dimension is not always a trustworthy index of volume change, and in later work the swell was calculated by comparing the volume of each specimen, as determined by displacement in water, before and after immersion.

In weighing the relative importance of the information obtained by the various tests in the Colorado investigation, it is believed that the data obtained by the immersion-compression test are the most significant in that, (1) the test utilized the whole aggregate instead of one fraction, (2) soundness of the aggregate is determined directly in terms of retention of stability of the entire mixture, (3) results are espressed in numerical values, unaffected by the personal factor involved in the estimation systems as used in the various stripping tests, and (4) the results are consistent with the service behavior of the materials in the road surface.

TESTS OF BITUMINOUS MIXTURES SHOW EFFECT OF VARYING TYPE AND QUANTITY OF FILLER

This method of test was considered so promising that it was used the following year in a laboratory study of the form and type of fillers commonly used in bituminous mixtures. Pulverized New York Avenue clay [2] was incorporated with Potomac River sand and gravel, both as a filler and as a coating on the surface of the particles of fine and coarse aggregate. In addition, mixtures were prepared in which limestone dust was used as the filler.

Two aggregate gradings were used in this work, ¾-inch-0 and ⅜-inch-0, as shown in table 6.

TABLE 6.—*Grading of aggregates used in the study of fillers*

Sieve size	Total amount passing— ¾-inch maximum size	Total amount passing— ⅜-inch maximum size	Sieve size	Total amount passing— ¾-inch maximum size	Total amount passing— ⅜-inch maximum size
	Percent	*Percent*		*Percent*	*Percent*
¾-inch	100		No. 14	39	68
½-inch	86		No. 30	31	45
⅜-inch	78	100	No. 50	19	23
No. 4	60	94	No. 100	12	16
No. 8	48	83	No. 200	9	10

In preparing the clay-coated aggregate, the clean aggregate was first moistened with water, then the pulverized clay was added and stirred until all particles seemed to be uniformly covered with a clay coating. The coated aggregate was then dried to a moisture-free condition, accompanied by constant stirring to minimize agglomeration. Bituminous material was then added. In the mixtures containing clay and limestone dust as fillers, the dry powders were mixed thoroughly with the dry aggregate before adding bituminous material.

[2] A plastic clay soil found in the vicinity of Washington, D. C., and designated as tuxedo clay by the Bureau of Plant Industry, U. S. Department of Agriculture.

TABLE 7.—*Characteristics of bituminous materials used*

Characteristics.	Type and grade of cut-back material: RC–4	MC–2	MC–3	Asphalt cement, 85–100 penetration
Specific gravity 77°/77° F.	0.985	0.966	0.978	1.018
Flash point, ° F.	120	135	160	304
Furol viscosity, at 140° F.		145.9	358	
Furol viscosity, at 180° F.	183			
Distillation:				
Distillate, percent by volume to 680° F.	14.5	24.5	22.0	
Tests on distillation residue:				
Penetration, 77° F., 100 gm., 5 sec.	97	194	191	85
Ductility, 77° F., 5 centimeters per minute, centimeters	208	122	127	110+
Total soluble in CCL_4, percent	99.91	99.92	99.88	
Total soluble in CS_2, percent				99.90
Oliensis spot test	Negative	Negative	Negative	

Three grades of bituminous material were used in this study, namely, 85–100 penetration asphalt cement, MC–3 cut-back and RC–4 cut-back asphalt, the more important characteristics of which are shown in table 7. The bitumen content of all mixtures was 6 parts bitumen to 100 parts of aggregate by weight.

Sufficient aggregate and bitumen for each test specimen were mixed by hand in individual batches. Mixing time was dictated by the ease or difficulty of coating the aggregate. In the case of the clay-coated aggregates some care in mixing was necessary, in order to avoid scuffing of the clay coating.

Mixing was done over a hot plate, and the following temperatures for batches representing the three types of bituminous material were not exceeded during the mixing process: Asphalt cement mixtures, 260° F.; MC–3, 120° F.; and RC–4, 160° F.

Test specimens of the asphalt-cement mixtures were molded immediately after mixing. Mixtures containing cut-back materials however were cured loose for 20 hours before molding—the batches with MC material in an oven at 140° F., and the batches with RC material at laboratory temperature. Mixtures with cut-back material were molded at air temperature.

Test specimens were cylinders, 4 inches in diameter and 4 inches in height, molded between double plungers under a load of 3,000 pounds per square inch. The molded specimens representing each combination of aggregate and bituminous material were cured in an oven at 140° F. for periods of 1 and 7 days. Some of the specimens representing each curing condition were tested in compression without water immersion while others were tested after immersion in water for 7 days. After removal from the water and just before testing, the swell of each specimen was noted. The platen speed of the testing machine was 0.2 inch per minute. An abstract of the test results is shown in table 8.

In general, the effect of immersion in water on the stability of the specimens was influenced by three factors, namely, the type of filler (plastic or nonplastic), the distribution of filler material (throughout the mixture or concentrated as a coating on the aggregate particles), and the characteristics of the bituminous material.

In all cases those specimens containing more than a slight amount of clay lost strength by a larger percentage than did those containing nonplastic filler. For the mixtures containing limestone-clay filler blends, the percentage loss was directly proportional to the percentage of clay contained. Specimens containing clay-coated aggregate showed the lowest percentage of re-

TABLE 8.—*Effect of immersion on stability of 4- by 4-inch specimens containing Potomac River sand and gravel in combination with plastic and nonplastic fillers.*

85-100 PENETRATION ASPHALT ¾-INCH MAXIMUM SIZE AGGREGATE

Quantity and type of filler	Air voids	Curing at 140° F.	Immersion	Swell	Compressive strength	Strength retained
	Percent	*Days*	*Days*	*Percent*	*Lb. per sq. in.*	*Percent*
9-percent limestone dust as filler	3.7	7	0		366	
	3.7	7	7	0	351	96
	4.9	1	0		408	
9-percent clay as filler		1	7	2.1	336	83
		7	0		460	
	4.9	7	7	1.4	369	80
8-percent clay as film on particles, 1-percent clay as filler	7.0	1	0		393	
	6.1	1	7	4.0	187	48
	7.0	7	0		418	
	6.6	7	7	3.2	243	58
RC-4 CUT-BACK MATERIAL ¾-INCH MAXIMUM SIZE AGGREGATE						
9-percent limestone dust as filler	3.3	1	0		152	
	4.1	1	7	0.5	166	109
	4.1	7	0		287	
	3.8	7	7	.6	257	90
9-percent clay as filler	3.8	1	0		247	
	4.1	1	7	2.6	146	59
	3.8	7	0		435	
		7	7	1.7	266	61
8-percent clay as film on particles, 1-percent clay as filler		1	0		232	
	4.6	1	7	6.3	66	28
		7	0		361	
	4.6	7	7	4.4	134	37
RC-4 CUT-BACK MATERIAL, ⅜-INCH MAXIMUM SIZE AGGREGATE						
10-percent limestone dust as filler	11.6	1	0		131	
	11.9	1	7	1.2	108	82
	11.9	7	0		199	
		7	7	.7	181	91
10-percent clay as filler	11.9	1	0		236	
		1	7	3.9	77	33
	12.4	7	0		311	
	12.4	7	7	3.6	162	52
3-percent clay and 7-percent limestone dust as filler	11.6	1	0		155	
	11.9	1	7	3.6	119	77
	11.2	7	0		256	
	11.2	7	7	1.4	183	71
10-percent clay as film on particles	13.6	1	0		257	
	13.2	1	7	7.0	31	12
	14.0	7	0		306	
	14.0	7	7	5.3	66	22
7-percent clay as film on particles, 3-percent limestone dust as filler	13.6	1	0		203	
	13.6	1	7	6.1	25	12
	13.6	7	0		282	
	13.2	7	7	5.2	41	15
3-percent clay as film on particles, 7-percent limestone dust as filler	11.9	1	0		155	
	11.9	1	7	2.5	60	39
	12.4	7	0		232	
		7	7	1.7	114	49
MC-3 CUT-BACK MATERIAL, ¾-INCH MAXIMUM SIZE AGGREGATE						
9-percent limestone dust as filler	4.9	1	0		65	
		1	7	0.2	63	97
		7	0		97	
	5.3	7	7	0	123	127
9-percent clay as filler	5.3	1	0		161	
	5.3	1	7	4.6	30	19
		7	0		220	
		7	7		70	31
8-percent clay as film on particles, 1-percent clay as filler	6.2	7	0		252	
	6.7	7	7	5.0	26	10

tained stability. It was found that the compressive strength of the dry specimens containing clay was higher than that of similar specimens containing limestone filler.

The influence of time of curing on loss of stability is seen by comparing the stability losses of specimens in each mixture group representing the two oven-curing periods.

In general, a very consistent relation is evident between the swell of the specimens and the proportions of clay. As the percentage of clay was increased, the volume change increased, with the greater changes indicated by the mixtures containing clay as a coating on the aggregate particles than in those containing clay as a filler.

SPECIMENS TESTED DRY WERE BROUGHT TO TEST TEMPERATURE IN THIS INSULATED AIR CHAMBER

A STUDY OF WIDER SCOPE PLANNED

The usefulness of an immersion-compression test having been demonstrated within the limits of the two investigations described above, it was decided to determine the value of the test more completely by a study of considerably larger scope. Bituminous mixtures were selected with the view of throwing light on the following:

(1) The advantage of this type of test over the less direct stripping test now in general use.

(2) The sensitivity of the test to minor variations in the hydrophilic properties of the aggregate.

(3) The usefulness of the test in evaluating various types of filler material.

(4) The usefulness of the test in indicating variations in the quality of dense mixtures.

(5) The usefulness of the test in indicating the effect on mixture quality of various grades of asphaltic material.

Three bituminous materials were used—an 85–100 penetration asphalt, an MC–3 cut-back asphalt, and an RC–4 cut-back asphalt. The more important characteristics of these materials are shown in table 7. In all mixtures the bitumen content was 6 parts of bitumen to 100 parts of aggregate, by weight.

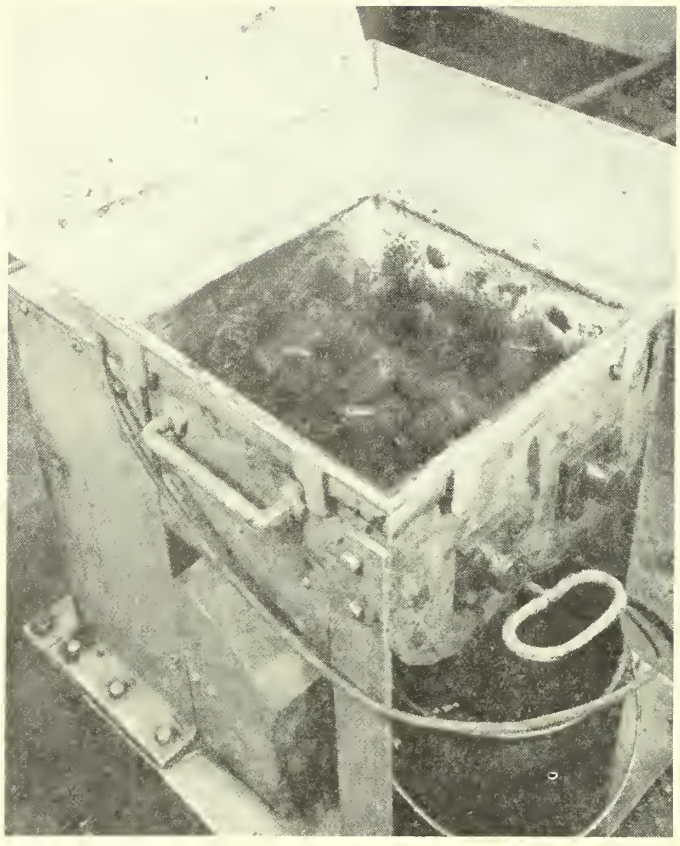
Four types of coarse aggregate were used—a crushed limestone (hydrophobic), a crushed quartzite (hydrophilic), a pit gravel (hydrophilic), and a river gravel (hydrophobic).

Three types of fine aggregate were used—river sand (hydrophobic), crushed limestone sand (hydrophobic), and crushed quartzite sand (hydrophilic).

TABLE 9.—*Characteristics of aggregates and fillers used*

Aggregates	Nature	Apparent specific gravity	Percent passing No. 200 sieve
Coarse aggregates:			
Crushed quartzite	Hydrophilic	2.63	
Crushed limestone	Hydrophobic	2.71	
Washed Potomac River gravel	do	2.64	
Washed pit gravel	Hydrophilic	2.64	
Fine aggregates:			
Washed Potomac River sand	Hydrophobic	2.64	2
Washed crushed quartzite	Hydrophilic	2.63	0
Washed crushed limestone	Hydrophobic	2.71	0
Fillers:			
Pulverized silica	Hydrophobic	2.64	100
Pulverized limestone	Hydrophobic	2.71	100
Pulverized clay [1]	Hydrophilic	2.67	100

[1] Liquid limit, 71; plasticity index, 46.



MIXTURES FOR COMPRESSION SPECIMENS WERE PREPARED IN A TWIN-PUGMILL MIXER.

TABLE 10.—*Composition of aggregate blends*

Passing	7-percent filler blends	12-percent filler blends	Passing	7-percent filler blends	12-percent filler blends
	Percent	*Percent*		*Percent*	*Percent*
½-inch sieve	100	100	No. 10 sieve	37	42
⅜-inch sieve	66	69	No. 200 sieve	7	12
No. 4 sieve	42	47			

Three types of filler were used—pulverized limestone, pulverized silica, and pulverized New York Avenue clay. All fillers passed a 200-mesh sieve.

Selected combinations of these materials, and not all possible combinations, were used in the investigation. Characteristics of aggregates and fillers are given in table 9.

Two percentages of filler were used with each combination of coarse and fine aggregate. The composition of the aggregate blends by sizes is shown in table 10.

All mixtures were prepared in a laboratory twin-pugmill mixer of 20 pounds capacity. The mixer has electric heating elements to prevent excessive cooling during mixing.

In general, the mixing time, measured from the time all the bituminous material had been added to the discharge of the batch, was 1½ minutes. Mixtures containing clay filler required up to 2½ minutes for the production of a batch of uniform appearance. Prior to mixing, the bituminous materials were heated to the following temperatures: Asphalt cement, 300° F.; MC–3, 200° F.; RC–4, 225° F.

In preparing the mixtures containing asphalt cement, the aggregate was heated to a temperature of 325° F. before mixing. In the cut-back mixtures the aggregate was unheated. Temperature of the asphalt cement mixtures at the time of molding was 260° F., and the molds were heated to that temperature also. Molding of the cut-back mixtures was done at room temperature.

TABLE 11.—*Results of modified Nicholson stripping test on coarse fractions of aggregates used in immersion-compression test study*

Type of aggregate	Estimated area coated after 45 minutes of rotation; aggregate mixed with—		
	AC (85–100)	RC–4	MC–3
Quartzite	100	90	80
Limestone	100	100	100
River gravel	100	100	95
Pit gravel	100	100	85

Molding of the asphalt cement mixtures followed mixing immediately. Molding of the MC–3 and RC–4 mixtures followed 24 hours of curing at room temperature. All specimens were 4 inches in diameter and 4 inches in height, and were molded between double plungers under a load of 3,000 pounds per square inch, held for 2 minutes.

THE TEST PROCEDURE USED IN THE EXTENDED STUDY DESCRIBED

After molding, all specimens were placed in an oven maintained at a constant temperature of 140° F. After 24 hours in the oven they were removed and allowed to stand overnight at room temperature. The second day after molding, the specimens were weighed in air and water for specific gravity and volume determinations. Three of the 12 specimens representing each mixture were then brought to test temperature (77° F.) and tested for compressive strength. At the same time, the other nine specimens were completely submerged in a water bath maintained at 77° F. They were tested for compressive strength in sets of three specimens at the end of 2, 4, and 7 days of immersion. Just prior to testing, each specimen was weighed in water, then surface dried and weighed in air, for the purpose of calculating absorption and volume change.

The compressive strength of all specimens was determined by means of a hydraulic testing machine, using a platen speed of 0.2 inch per minute, the specimens being free of lateral support. The results of the modified Nicholson test are given in table 11. The results of the immersion-compression test, and swell and absorption data, are given in table 12.

Concurrently with this study, a laboratory investigation was carried on to determine the effect of several additives in preventing stripping of bituminous coatings from aggregates. In addition to the Nicholson test, the immersion-compression test was used to measure the effect of these additives on the retention of stability by the mixtures. Although this work has not yet been completed, a portion of the data is included in this report as being of interest in further demonstrating the usefulness of the immersion-compression test. These data are shown in table 13.

RESULTS DEMONSTRATE USEFULNESS OF IMMERSION-COMPRESSION TEST IN DETERMINING EFFECT OF WATER ON MIXTURES

The use of selected aggregate gradings resulted in very high densities in the molded specimens. Regardless of the type of coarse aggregate used, the mixtures containing 12-percent limestone dust were denser than those containing the same quantity of either of the other fillers. In the mixtures with 7-percent filler, however,

TABLE 12.—*Results of immersion-compression tests on specimens containing hydrophilic and hydrophobic aggregates*

ASPHALT CEMENT (85–100)

Time of immersion	QUARTZITE AND POTOMAC RIVER SAND								LIMESTONE AND POTOMAC RIVER SAND							
	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained
Days	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*
Silica dust, percent	7				12				7				12			
Air voids, percent[1]	4.4				3.0				4.0				3.3			
None							348				261				295	
None			252				317				268				308	
None			278				346				246				306	
Average			265				337				258				303	
2	1.1	0.5	324		0.9	0.8	347		1.1	0.4	279		0.1	0.4	344	
2	1.0	.5	322		.3	.4	391		.1	.6	288		1	.4	352	
2	.7	.4	352		.3	.3	374		.1	.4	312		.2	.5	353	
Average	.9	.5	333	126	.5	.5	371	110	.4	.5	293	114	.1	.4	350	116
4	.9	.5	364		.2	.5	344		.2	.6	300		.4	.5	369	
4	1.0	.7	296		.2	.4	342		.6	.9	303		.1	.6	386	
4	1.2	.7	345		.7	.7	333		.1	.6	325		.5	.8	370	
Average	1.0	.6	335	126	.4	.5	340	101	.3	.7	309	120	.1	.6	375	123
7	.7	1.0	267		.1	.4	337		.5	.7	295		.3	.5	333	
7	1.5	1.4	259		.5	.8	335		.6	.9	269		.2	.5	343	
7	1.2	1.3	257		.7	.9	341		1.2	1.1	284		.4	.6	338	
Average	1.1	1.2	261	98	.4	.7	338	100	.8	.9	283	110	.3	.5	338	112
Limestone dust, percent	7				12				7				12			
Air voids, percent[1]	3.5				2.6				3.7				2.0			
None							343				265				318	
None			277				349				253				299	
None			298				361				267				322	
Average			288				351				262				313	
2	1.2	0.6	338		0.4	0.5	357			0.4	308			0.3	378	
2	.7	.3	329		.2	.3	367			.4	318			.2	354	
2	.6	.2	346		.2	.4	365			.5	303			.3	372	
Average	.8	.4	338	117	.3	.4	363	103	0.1	.4	310	118	0.5	.3	368	118
4	1.4	.8	333		.5	.6	332			.4	287			.2	329	
4	1.5	.8	329		.7	.6	329			.5	266			.3	345	
4	.7	.6	285		.2	.4	331			.5	296			.4	342	
Average	1.2	.7	316	110	.5	.5	331	94	0	.5	283	108	.5	.3	339	108
7	.5		255		.1	.2	347		.4	.9	243		.1	.4	312	
7	.5	.8	269		.1	.3	319		.5	1.0	243		0	.5	306	
7	1.0	.8	263		.2	.5	328		1.5	1.5	241		0	.3	307	
Average	.7	.8	262	91	.1	.3	331	94	.8	1.1	242	92	0	.4	308	98
Clay dust, percent	7				12				7				12			
Air voids, percent[1]	3.1				3.2				2.5				2.6			
None			313				410				305				399	
None			298				404				344				408	
None			316				405				319				453	
Average			309				406				323				420	
2	0.2	0.5	329		0.5	0.7	395		0.2	0.4	377		0.2	0.5	421	
2	0	.6	330		1.0	1.0	384		.2	.5	372		6	7	427	
2	.6	.7	342		.9	1.0	382		.6	.6	378		4	6	427	
Average	.3	.6	334	108	.8	.9	387	95	.3	.5	376	116	4	6	426	101
4	.5	.6	349		.6	.8	403		.2	.5	370		8	7	450	
4	.5	.7	343		1.0	1.1	397		.2	.6	341		4	8	454	
4	1.0	1.1	330		2.1	1.5	356		.5	.6	363		9	9	442	
Average	.7	.8	341	110	1.2	1.1	385	95	.3	.6	358	111	7	8	449	107
7	.5	.6	349		.8	1.0	354		.4	.5	341		8	7	387	
7	.8	.8	341		1.4	1.2	366		.4	.6	308		14	8	111	
7	1.1	1.0	335		1.5	1.3	353		.5	.7	327		5	9	389	
Average	.8	.8	342	111	1.2	1.2	358	88	.4	.6	325	101	8	8	336	94

See footnotes at end of table.

TABLE 12.—*Results of immersion-compression tests on specimens containing hydrophilic and hydrophobic aggregates*—Continued

RC-4 CUT-BACK ASPHALT

Time of immersion	QUARTZITE AND POTOMAC RIVER SAND								LIMESTONE AND POTOMAC RIVER SAND							
	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained
Days	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*
Silica dust, percent	7				12				7				12			
Air voids, percent [1]	3.8				2.7				2.6				2.2			
None			110				108				105				128	
None			107				118				108				123	
None			111				111				106				124	
Average			109				112				106				125	
2	0.6	0.9	96		0.6	0.9	96		0.3	0.6	95		0.8	0.9	106	
2	.5	1.0	96		.6	1.0	97		.3	.7	84		.8	1.0	112	
2	.4	.9	101		.5	.8	92		.2	.6	87		.9	.9	110	
Average	.5	.9	98	90	.6	.9	95	85	.3	.6	89	84	.8	.9	109	87
4	.6	1.1	76		2.0	1.7	73		.3	1.0	78		1.3	1.1	97	
4	.7	1.1	86		1.8	1.5	76		.8	1.0	74		.9	1.1	97	
4	.5	1.2	82		2.0	1.7	70		.7	.9	78		1.1	1.1	100	
Average	.6	1.1	81	74	1.9	1.6	73	65	.6	1.0	77	73	1.1	1.1	98	78
7	1.3	1.4	72		5.4	3.2	25		1.2	1.4	61		1.7	1.5	72	
7	1.3	1.6	72		4.8	2.8	32		1.6	1.4	57		2.0	1.6	63	
7	1.1	1.6	74		4.3	2.7	39		1.8	1.3	57		1.8	1.4	70	
Average	1.2	1.5	73	67	4.8	2.9	32	29	1.5	1.4	58	55	1.8	1.5	68	54
Limestone dust, percent	7				12				7				12			
Air voids, percent [1]	3.4				2.3				2.1				1.5			
None							116				101				104	
None			109				121				105				105	
None			105				111				101				102	
Average			107				116				102				104	
2	0.3	0.8	95		0.3	0.6	103		0.6	0.6	101		0.5	0.6	107	
2	0	.7	95		.3	.6	112		.6	.6	104		.3	.6	112	
2	.5	1.0	85		.1	.7	113		.6	.7	96		.4	.6	105	
Average	.3	.8	92	86	.2	.6	109	94	.6	.6	100	98	.4	.6	108	104
4	.7	1.1	87		.5	.7	98		.6	.8	98		.6	.9	112	
4	.4	1.0	90		.5	.9	110		.5	.7	103		.4	.6	105	
4	.6	1.1	87		.3	.9	107		.6	.8	99		.5	.6	112	
Average	.6	1.1	88	82	.4	.8	105	90	.6	.8	100	98	.5	.7	110	106
7	.7	1.2	79		.5	1.0	94		.4	.8	93		.3	.7	106	
7	.8	1.1	81		.7	1.0	89		.5	.8	100		.5	.7	105	
7	.4	1.2	89		1.0	1.3	89		.5	.8	99		.4	.6	103	
Average	.6	1.2	83	78	.7	1.1	91	78	.5	.8	97	95	.4	.7	105	101
Clay dust, percent	7				12				7				12			
Air voids, percent [1]	2.6				3.1				1.5				1.8			
None			171				251				142				219	
None			180				252				152				242	
None			182				259				147				249	
Average			178				254				147				237	
2	1.0	1.0	140		1.4	1.7	163		1.3	1.0	107		2.2	1.7	127	
2	1.1	1.2	142		1.4	1.5	175		1.0	.8	104		1.9	1.5	139	
2	.8	1.1	145		1.8	1.8	161		.9	.9	117		2.9	2.1	109	
Average	1.0	1.1	142	80	1.5	1.7	166	65	1.1	.9	109	74	2.3	1.8	125	53
4	1.0	1.1	134		2.3	2.1	139		1.7	1.2	92		3.7	2.2	85	
4	1.4	1.4	126		2.3	2.0	143		1.7	1.2	97		3.8	2.2	83	
4	1.0	1.2	135		3.4	2.6	125		2.0	1.3	95		6.1	2.9	67	
Average	1.1	1.2	132	74	2.7	2.2	136	54	1.8	1.2	95	65	4.5	2.4	78	33
7	1.9	1.7	109		3.5	2.6	99		2.9	1.7	64		10.6	5.0	19	
7	1.9	1.6	111		3.3	2.6	103		2.6	1.6	68		10.2	4.9	8	
7	2.2	1.8	109		4.8	3.4	84		2.7	1.6	64		12.2	6.1	11	
Average	2.0	1.7	110	62	3.9	2.9	95	37	2.7	1.6	65	44	11.0	5.3	13	5

See footnotes at end of table.

TABLE 12.—*Results of immersion-compression tests on specimens containing hydrophilic and hydrophobic aggregates*—Continued

MC-3 CUT-BACK ASPHALT

Time of immersion	QUARTZITE AND POTOMAC RIVER SAND								LIMESTONE AND POTOMAC RIVER SAND							
	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained
Days	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*
Silica dust, percent / **Air voids, percent** [1]	7 / 3.7				12 / 3.1				7 / 2.4				12 / 1.4			
None			34				49				36				51	
None			25				53				39				47	
None			26				47				32				53	
Average			32				50				36				50	
2	0.8	1.1	31		1.0	1.2	48		0.2	0.5	29		0.5	0.6	43	
2	.8	1.2	37		.4	.9	50		.5	.6	32		.3	.5	47	
2	.9	1.3	30		1.2	1.4	49		.4	.6	31		.6	.6	42	
Average	.8	1.2	33	103	.9	1.2	49	98	.4	.6	31	86	.5	.6	44	88
4	1.5	1.6	21		2.4	2.0	27		.3	.9	24		2.2	1.3	27	
4	1.2	1.5	21		1.8	1.6	33		.7	.8	24		1.9	1.2	27	
4	1.4	1.5	21		1.9	1.8	29		.8	.9	25		2.2	1.4	23	
Average	1.4	1.5	21	66	2.0	1.8	30	60	.6	.9	24	67	2.1	1.3	26	52
7	[2]	[2]	[2]		6.1	3.5	7		2.6	1.8	14		7.3	3.5	8	
7	2.7	2.2	12		5.9	3.5	8		2.1	1.6	15		[2]	[2]	9	
7	2.9	2.3	11		5.1	2.9	11		2.4	1.7	17		10.0	4.6	11	
Average	2.8	2.2	12	38	5.7	3.3	9	18	2.4	1.7	15	42	8.6	4.0	9	18
Limestone dust, percent / **Air voids, percent** [1]	7 / 3.3				12 / 2.5				7 / 1.7				12 / 0.8			
None			47				51				39				41	
None			32				51				34				45	
None			37				49				35				43	
Average			39				50				36				43	
2	0.2	0.8	37		0.2	0.4	55		0	0.3	34		0	2.0	50	
2	.2	.7	35		.2	.6	51		0	.2	36		0	1.9	42	
2	.2	.8	37		.4	.5	52		0	.2	35		0.1	1.6	46	
Average	.2	.8	36	92	.3	.5	53	106	0	.2	35	97	0	1.8	46	107
4	.6	1.0	33		.1	.6	51		0	.4	40		0	2.0	46	
4	.6	1.0	34		.7	.8	43		0	.4	33		0	2.3	45	
4	.5	1.0	35		.2	.7	44		0	.4	37		0	2.0	50	
Average	.6	1.0	34	87	.3	.7	46	92	0	.4	37	103	0	2.1	47	109
7	.7	1.2	28		.2	.7	50		0	.5	35		0	2.2	45	
7	.6	1.2	32		.3	.8	44		0	.4	30		0	2.2	47	
7	.5	1.0	34		.6	.9	50		0	.4	34		0	2.8	44	
Average	.6	1.1	31	79	.4	.8	48	96	0	.4	33	92	0	2.4	45	105
Clay dust, percent / **Air voids, percent** [1]	7 / 2.8				12 / 2.5				7 / 1.1				12 / 1.5			
None			72				127				66				93	
None			72				125				67				96	
None			70				127				66				95	
Average			71				126				66				95	
2	0.2	0.6	62		1.6	1.3	89		0.4	0.5	53		0.2	0.7	49	
2	.2	.6	58		1.2	1.2	90		.6	.5	57		1.7	1.2	51	
2	.4	.6	64		1.6	1.4	83		.7	.5	53		1.2	.9	55	
Average	.3	.6	61	86	1.5	1.3	87	69	.6	.5	54	82	1.0	.9	52	55
4	1.1	1.1	43		1.8	1.4	71		1.2	.8	37		4.5	2.2	28	
4	1.3	1.0	42		2.1	1.6	67		1.3	.8	41		4.1	2.0	28	
4	1.2	1.1	41		1.7	1.5	74		1.0	.9	37		4.7	2.3	26	
Average	1.2	1.1	42	59	1.9	1.5	71	56	1.2	.8	38	58	4.4	2.2	27	28
7	2.0	1.4	33		3.4	2.3	50		2.3	1.2	29		13.3	5.4		
7	1.9	1.5	33		1.9	2.1	49		2.7	1.4	25		17.6	10.9	10	
7	2.2	1.6	35		2.5	1.8	56		3.3	1.7	21		19.4	11.2	9	
Average	2.0	1.5	34	48	2.6	2.1	52	41	2.8	1.4	25	38	16.8	9.2	6	6

See footnotes at end of table.

TABLE 12.—*Results of immersion-compression tests on specimens containing hydrophilic and hydrophobic aggregates*—Continued

MC-3 CUT-BACK ASPHALT

Time of immersion	PIT GRAVEL AND POTOMAC RIVER SAND				POTOMAC RIVER GRAVEL AND POTOMAC RIVER SAND				CRUSHED QUARTZITE AND QUARTZITE SAND				CRUSHED LIMESTONE AND LIMESTONE SAND			
	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained	Swell	Water absorbed	Compressive strength	Strength retained
Days	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*	*Percent*
Silica dust, percent 12 **Air voids, percent [1] 3.9**					**12 3.4**				**12 3.3**				**12 1.4**			
None			45				37				66				62	
None			54				39				68				62	
None			43				33				70				63	
Average			47				36				68				62	
2	1.6	1.6	33		0.9	1.2	38									
2	1.2	1.5	37		.8	1.1	38									
2	1.0	1.4	35		.8	1.0	38									
Average	1.3	1.5	35	74	.8	1.1	38	106								
4	4.4	2.9	10		1.7	1.6	24		7.3	4.0	8		3.2	1.7	33	
4	4.0	2.8	11		1.1	1.4	25		5.7	3.3	14		3.1	1.7	35	
4	3.6	2.6	14		2.0	1.7	23		6.0	3.3	11		3.0	1.6	33	
Average	4.0	2.8	12	26	1.6	1.6	24	67	6.3	3.5	11	16	3.1	1.7	34	55
7	[2]	[2]	[2]		4.7	3.0	8		[2]	[2]	[2]		6.4	3.0	16	
7	[2]	[2]	[2]		3.6	2.6	13		[2]	[2]	[2]		7.1	3.4	13	
7	[2]	[2]	[2]		4.1	2.6	10		[2]	[2]	[2]		8.0	3.8	16	
Average				0	4.1	2.7	10	28				0	7.2	3.4	15	24
Limestone dust, percent 12 **Air voids, percent [1] 3.4**					**12 2.5**				**12 2.9**				**12 1.3**			
None			44				43				70				62	
None			45				42				69				60	
None			41				43				64				56	
Average			43				43				68				59	
2	0.7	1.1	45		0	0.4	42									
2	.2	.9	41		0.4	.2	43									
2	.4	.9	46		.3	.3	45									
Average	.4	1.0	44	102	.2	.3	43	100								
4	.6	1.1	41		.3	.6	38		0.3	0.9	56		0	0.3	72	
4	.6	1.0	47		0	.4	45		.3	.8	56		0	.3	73	
4	.8	1.1	42		.1	.2	44		0	.7	60		0	.3	69	
Average	.7	1.1	43	100	.1	.4	42	98	.2	.8	57	84	0	.3	71	120
7	.8	1.2	34		.1	.7	40		.5	1.0	51		0	.4	64	
7	.9	1.1	41		0	.6	44		.7	1.0	47		.2	.4	62	
7	1.1	1.4	33		.1	.6	41		.1	.8	57		0	.4	60	
Average	.9	1.2	36	84	.1	.6	42	98	.4	.9	52	76	.1	.4	62	105
Clay dust, percent 12 **Air voids, percent [1] 3.7**					**12 3.1**				**12 2.3**				**12 1.5**			
None			129				108				134				111	
None			121				120				131				114	
None			120				113				136				109	
Average			123				114				134				111	
2	3.7	2.8	45		1.0	1.2	77									
2	2.6	2.2	49		.7	1.0	90									
2	2.7	2.5	48		.4	.9	88									
Average	3.0	2.5	47	38	.7	1.0	85	74								
4	5.9	3.5	24		2.3	2.0	53		3.4	1.9	58		6.7	2.9	24	
4	4.3	2.8	32		2.3	2.1	53		3.8	2.1	51		8.1	3.5	21	
4	5.6	3.4	26		1.6	1.7	60		4.3	2.3	45		9.2	3.9	19	
Average	5.3	3.2	27	22	2.1	1.9	55	48	3.8	2.1	51	38	8.0	3.4	21	19
7	13.5	6.4	10		5.9	3.8	22		13.5	5.9	13		21.0	8.8	8	
7	14.2	6.8	8		4.1	2.7	35		10.9	4.7	22		[2]	[2]	[2]	
7	12.2	5.8	10		4.0	2.6	32		[2]	[2]	[2]		[2]	[2]	[2]	
Average	13.3	6.3	9	7	4.7	3.0	30	26	12.2	5.3	12	9			3	3

[1] Air voids in compacted specimens.

[2] Specimen disintegrated in water bath.

the clay filler produced the highest density. As between the stone mixtures and the gravel mixtures, the gravel mixtures were less dense. Mixtures containing crushed limestone produced higher densities than those containing crushed quartzite, when comparable percentages of the three respective fillers were used.

As would be expected, the stabilities of dry specimens representing all aggregate and filler groups reflected directly the consistency of the bituminous material used. That is, for any type of coarse aggregate containing the same percentage of any of the three fillers, stabilities were highest in specimens containing asphalt cement, lowest in those containing MC–3 material, with intermediate values for those containing RC–4 material.

For any given grade of bituminous material and for comparable percentages and types of filler, the mixtures containing quartzite as coarse aggregate generally had higher stabilities than those containing limestone. This may be explained by the rougher surface texture of the quartzite. In the mixture groups containing limestone and quartzite as coarse aggregate, increasing the percentage of filler from 7 to 12 invariably resulted in stability increases in the dry specimens ranging from as low as 2 percent for mixtures of crushed limestone, RC–4 material, and limestone filler to 77 percent for the mixtures of quartzite, MC–3 material, and clay filler. Dry specimens containing clay filler had higher stabilities than comparable specimens containing either limestone or silica fillers.

It was observed that in many of the mixtures containing asphalt cement the specimens that had been immersed in water had higher stability values than corresponding dry specimens. In future work with such mixtures, a period of immersion in water in excess of 7 days will be used, with the expectation of developing more rational information.

This same phenomenon was also observed in several of the mixtures with MC–3 and RC–4 materials after 2 days immersion. Since it does not appear that the character of the aggregates, of itself, is responsible for this apparent increase in stability, the values from specimens immersed only 2 days will be disregarded in discussing the data. Results of the tests on mixtures containing the MC and RC materials, expressed in terms of percentage of stability retained after immersion, are shown graphically in figures 1, 2, and 3.

The effect of immersion in water on mixtures with RC–4 material is shown in figure 1, the data being from table 12. For easy comparison, the curves representing mixtures containing quartzite and the three fillers and those containing limestone and the three fillers have been arranged side by side. Considering first the effect of the character of the filler only on the quartzite mixtures, it is apparent that 7-percent clay produces a greater loss in stability than the same percentage of either of the other fillers, with the 7-percent limestone dust mixture showing the least reduction. With increased percentage of the respective fillers, the greatest loss is produced by the silica, and the least loss again is shown by the limestone dust.

In the case of the limestone mixtures with either percentage of fillers limestone filler produced little or no loss with the clay filler showing the greatest loss.

The stabilities of the quartzite and limestone mixtures containing comparable percentages of the three fillers are not entirely in accord with what would be expected from the results of the stripping tests on the coarse aggregates. It is only in the mixtures containing limestone filler that results consistent with the stripping properties of the coarse aggregates were obtained. Mixtures containing limestone and limestone filler in either percentage weakened considerably less than those containing quartzite stone and limestone filler. On the other hand, 7 percent of silica dust in combination with quartzite stone showed less loss than a corresponding percentage of silica dust with limestone coarse aggregate. However, the loss shown by 12 percent of silica dust with quartzite was considerably greater than that shown by the comparable mixture in which limestone was used as the coarse aggregate. It is possible that the disparity in proportional effect as between the two percentages of silica filler in combination with the two coarse aggregates may be due to experimental error. Comparison of the quartzite-clay and the limestone-clay mixtures shows that clay produces much greater loss with limestone than with quartzite.

The stability retained by mixtures of quartzite, river sand, and the three fillers combined with MC–3 material in relation to that of corresponding mixtures in which the coarse aggregate is limestone, is also shown in figure 1. The data are from table 12. Considering only the effect of the various fillers, in the quartzite mixtures the highest loss was produced by the silica filler and the lowest by the limestone filler. In the limestone mixtures however clay produced the greatest loss with limestone dust the least. Comparing quartzite with limestone mixtures containing corresponding types of filler, it is seen that the hydrophobic property of the limestone did contribute slightly to the retention of stability by the mixtures containing silica or limestone fillers but this is not true of the mixtures containing clay.

The effect of water on mixtures containing gravel and the three fillers is seen in figure 2, based on the data of table 12. As stated previously, the Potomac River gravel is considered to be hydrophobic and the pit gravel to be hydrophilic. Results of the immersion-

TABLE 13.—*Effect of additives in reducing loss of strength in 4- by 4-inch cylinders as indicated by immersion-compression and stripping tests*

Additives used with MC–2 [1] cut-back asphalt	Coarse aggregate (½-inch–No. 10) [2]	Water absorbed after 4 days in water	Volumetric swell after 4 days in water [3]	Compressive strength		Strength retained after 4 days in water	Stripping test,[5] area coated
				Dry [4]	After immersion 4 days in water		
		Percent	*Percent*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*	*Percent*
None	Trap	------	------	57	0	0	80
	Granite	3.8	4.6	62	17	27	60
	Limestone	1.6	1.7	56	39	70	100
Additive A, 1-percent	Trap	2.0	0	60	62	103	95
	Granite	1.5	.6	59	57	97	95
	Limestone	.8	.3	50	56	112	100
Additive B, 0.65-percent	Trap	2.0	1.0	65	45	69	100
	Granite	1.6	1.1	64	53	83	100
	Limestone	.6	0	51	60	118	100
Additive C, 0.5-percent	Granite	1.9	1.4	62	43	69	95
	Limestone	.8	.1	49	53	108	100
Additive D, 1-percent	Granite	1.5	.6	59	57	97	95
	Limestone	.8	.3	50	56	112	100
Additive E, 2-percent	Granite	1.4	.4	61	55	90	95
	Limestone	.6	0	49	51	104	100
Additive F, 2-percent	Granite	1.4	1.1	66	49	75	90
	Limestone	.7	.2	48	49	102	95

[1] Important characteristics shown in table 7.
[2] Composition of mixture: Coarse aggregate, 60 percent; fine aggregate, river sand, 30 percent; filler, silica, 10 percent.
[3] Cylinders immersed in water at 77° F. after curing.
[4] Cylinders tested after curing for 24 hours at 140° F.
[5] Modified Nicholson test on coarse aggregate, condition at end of 45 minutes.

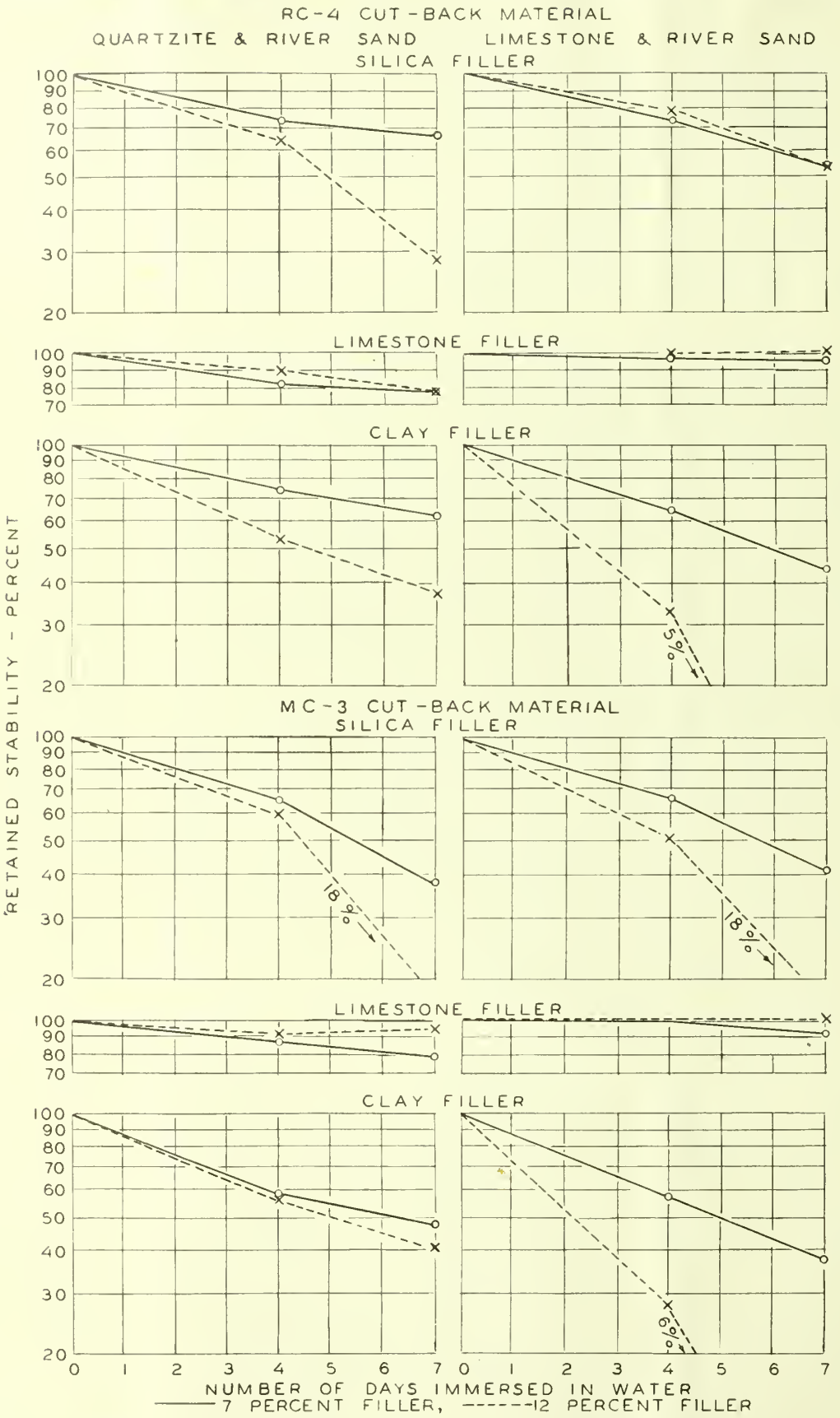


FIGURE 1.—STABILITY RETENTION OF SPECIMENS CONTAINING CRUSHED STONE.

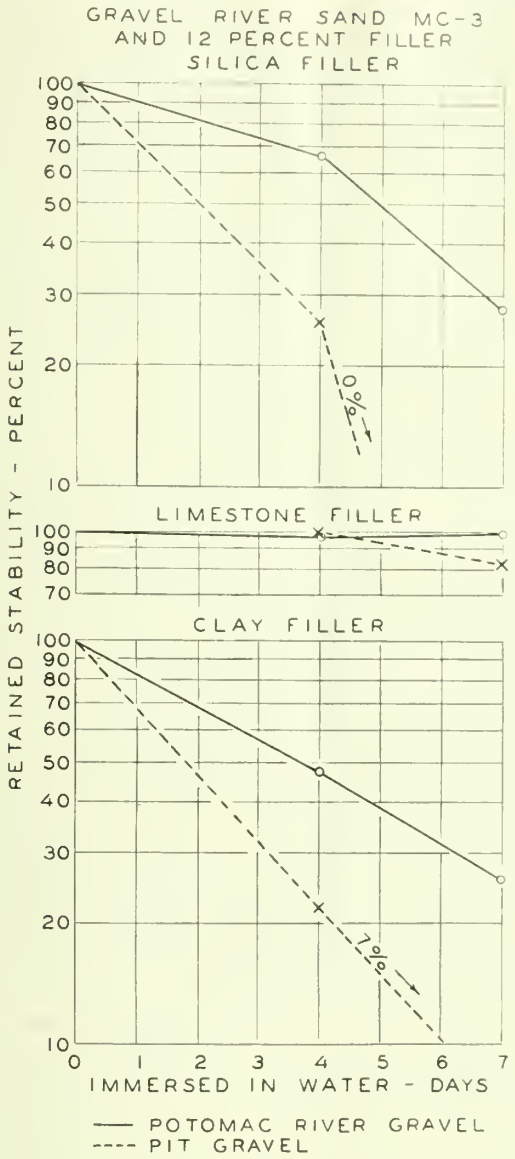


FIGURE 2.—STABILITY RETENTION OF SPECIMENS CONTAINING GRAVEL.

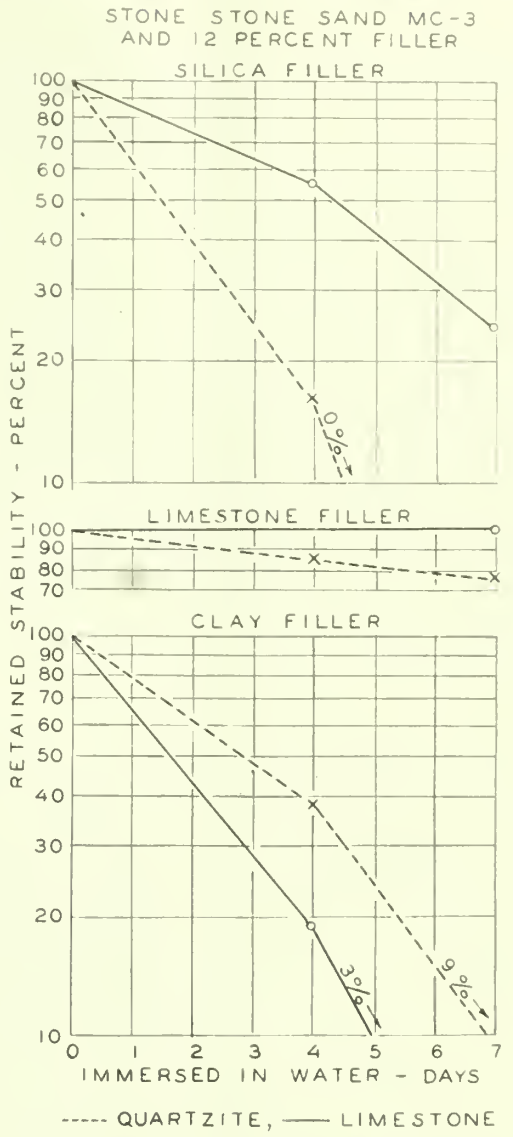


FIGURE 3.—STABILITY RETENTION OF SPECIMENS CONTAINING CRUSHED STONE SAND FRACTION.

compression test seem to corroborate this distinction. However, judging from the test results on the mixtures with limestone filler the effect of variation in the coarse aggregate fraction is comparatively small. Different types of filler, however, cause large differences in rates of loss of stability. In pit gravel mixtures silica filler produced the greatest loss and limestone filler the least, while in the Potomac River gravel mixtures the losses at 7 days for silica and clay fillers were only slightly different.

In most of the mixtures studied, the fine aggregate consisted of washed Potomac River sand, which is considered to be hydrophobic, although it is difficult if not impossible to evaluate this property in fine aggregate fractions with the commonly used stripping tests.

In order to obtain a more positive indication of the influence of the nature of the fine aggregate, mixtures were made in which the fine aggregate was prepared by crushing the same stone that composed the coarse aggregate. The data from tests of the mixtures are shown in table 12 and figure 3. In the quartzite group, the mixtures containing silica filler disintegrated completely in the water bath before the 7-day immersion period had elapsed. With quartzite aggregate the clay-filler mixtures had lost most of their stability at the end of 7 days immersion, while those with limestone filler lost 24 percent. In the limestone group, water caused most damage to the clay-filler specimens, less to the silica-filler mixtures, and none at all to the limestone-filler mixtures.

From the meager data of these exploratory tests, no definite conclusions regarding the effect of the character of the sand fraction can be drawn. Direct comparison of mixtures containing crushed stone sand and those containing river sand is not possible due to the impracticability of exactly duplicating the grading of the river sand by crushing stone. Gradings representative of these sands are shown in table 14.

In the study of antistripping additives with particular aggregates entirely different indications were implied by results of the stripping test and the immersion-compression test (table 13). For example, in the tests of mixtures in which no additives were used, the Nicholson test showed limestone to be least affected by water and trap rock to be next best. In the immersion-compression test, while the limestone mixture lost only 30

TABLE 14.—*Gradation of sand fractions*

Sieve No.	Potomac River sand	Quartzite	Limestone
	Percent passing	*Percent passing*	*Percent passing*
4	100	100	100
10	99	99	100
20	90	49	39
30	77	36	25
40	58	26	15
50	34	18	9
80	16	8	3
100	12	6	2
200	2	0	0

percent of its strength, the trap-rock mixtures lost all stability after 4 days in water.

As another example, in the tests with additive B, the Nicholson test showed all aggregates to be completely coated, whereas the immersion-compression test shows considerable difference in the behavior of mixtures containing each of the three stones. Also, the Nicholson test indicates that all five additives produced essentially equal improvement of coating on the granite, whereas the immersion-compression test shows that, while all the additives improved the retention of stability of the granite mixtures, the degree of improvement effected by the several additives differed considerably.

SUMMARY

Results of the studies described in this report indicate that an immersion-compression test, carried out in conformity with the suggested procedure, will produce information valuable in various phases of bituminous mixture design. The test is useful in:

1. Determining the effect of the use of hydrophilic aggregate, whether coarse or fine, on the stability of the mixture when exposed to the action of water. It is impractical to use the usual stripping tests in evaluating sands or fillers, yet results of the foregoing studies show the importance of fillers in the behavior of the whole mixture. It has been shown that satisfactory retention of stability may be developed in mixtures containing hydrophilic aggregate, provided suitable filler is used.
2. The selection of fillers with regard to their contribution to retention or loss of stability of the mixture in the presence of water.
3. The detection of aggregates that are structurally unsound when wet.
4. Determining the usefulness of additives in causing resistance to stripping of bituminous films from aggregate particles by water action.
5. Determining the minimum proportions of imported hydrophobic material required to reduce stability loss in the presence of water, in cases where local aggregate is found to be hydrophilic.
6. Testing aggregates considered for use in related types of construction, such as surface treatments. Although the primary value of the test is in the solution of problems in design of bituminous mixtures, it is believed that adaptations of the test would be useful in other bituminous work.

DETAILS OF TEST PROCEDURE

A complete description of a suggested test procedure follows:

1. The bituminous mixture for use in the test shall be composed of the aggregates, filler, and bituminous material in the proportions proposed for use in the actual construction. This is important, for it has been demonstrated that variations in aggregate, filler, or bituminous material result in corresponding differences in loss of stability.
2. Hot mixtures shall be molded immediately after mixing. All other mixtures shall be cured loose in air for 24 hours before molding. It has been found that such a curing period is necessary in order to obtain consistent results.
3. Cylindrical specimens 4 inches in diameter by 4 inches in height shall be molded under a load of 3,000 pounds per square inch by the double plunger method. Other means of compaction that will produce equally satisfactory results may be used. At least six specimens shall be molded for each mixture to be tested.

There is considerable diversity of opinion as to the proper relation of height to diameter of cylindrical specimens for compression testing. Some investigators hold that the influence of end effects may not be eliminated by the use of height-diameter ratios less than 2 to 1. Other workers report entirely consistent results with lower ratios when the specimens are tested between rubber disks. The importance of end effect in certain types of testing where absolute values are required is unquestioned, but, it is believed that for immersion-compression testing, where results are expressed in percentage of stability loss, the question as to the influence of end effects may be waived in the interest of convenience and practicability. In the development of the test procedure, several specimens 4 inches in diameter and 8 inches in height were tested under the same conditions and at the same time as other specimens of the same diameter and 4 inches in height. Although indicated compressive strengths of the two sets of specimens differed considerably, the percentage losses in stability checked quite well. The larger specimens were difficult to mold, and were much more susceptible to damage in the necessary handling incident to weighing and testing.

On the other hand, a specimen smaller than 4 inches by 4 inches would not be satisfactory for mixtures containing coarse aggregates.

4. All molded specimens shall be cured for 24 hours in an oven maintained at a uniform temperature of 140° F.

Freshly mixed and compacted roadway surfaces are immediately subjected to water action only in exceptional cases. Immersion of freshly molded specimens in the laboratory would result in greatly exaggerated effects.

5. After oven curing, the specimens shall be allowed to cool to laboratory temperature, then weighed in air and water for density and volume determinations.
6. Three of the six specimens shall then be tested in compression at room temperature after having been brought to a temperature of 77° F. in an air bath. The other three specimens shall be submerged in water at a temperature of 77° F. for a period of 4 days, after which they shall be removed from the water, weighed in air and water for moisture and volume determinations, and tested in compression.

In the work reported here, the specimens were placed on glass or aluminum plates in the water bath. These plates were found to be very convenient in preventing spalling or distortion in handling those specimens that became softened in the water bath.

Immersion for 4 days is recommended as productive of consistent and significant results in testing mixtures containing liquid asphalt materials. In testing hot mixtures, a longer immersion period will be required.

7. All specimens shall be tested in compression without lateral confinement, and at a rate of vertical deformation of 0.2 inch per minute.

This testing speed is in conformity with previous work of a similar nature.

It will be necessary to obtain much more data correlating service behavior with laboratory test results before test limits can be assigned for specification purposes. However, based on the work already done, it is believed that mixtures retaining not less than 75 percent of the dry stability after immersion for 4 days may be expected to have satisfactory resistance to water in service.

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ADDITIVES FOR BITUMINOUS MATERIALS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by PAUL F. CRITZ, Senior Highway Engineer, and JOSEPH F. GOODE, Associate Highway Engineer

THE DETRIMENTAL EFFECT OF MOISTURE upon the service behavior of bituminous surfaces has long been a concern of engineers and, recently, attempts have been made to determine the basic causes of unsatisfactory behavior and to develop remedies. On this subject there is rather general agreement that:

(*a*) Aggregates vary considerably in their resistance to coating with bituminous materials and in their resistance to the loss of such coating in the presence of moisture.

(*b*) These variations are due to the character of both the aggregates and the bituminous materials.

(*c*) Chemical agents can be used to obtain a coating of bitumen and to increase its resistance to stripping.

PROPERTIES OF MATERIALS USED IN TESTS DESCRIBED

A number of chemical agents are being produced commercially and offered as aids in coating wet aggregate and in coating and the retention of coating on aggregates that normally may be adversely affected by moisture. To determine the extent to which commercial products are beneficial a study has been made of a number of products to obtain definite information on the following:

1. Effect of additives in coating wet aggregates with bitumen.
2. Effect of additives in reducing film stripping.
3. Effect of varying percentages of additives.
4. Effect of additives upon the characteristics of the bituminous materials with which they are used.
5. Effect of additives when used with different bituminous materials.
6. Effect of additives in reducing loss of strength of bituminous mixtures due to the presence of moisture.
7. Permanency of benefit obtained by the use of additives.

No attempt has been made to include in this investigation all of the materials that may be available. There are included, however, a number of representative materials. They are designated in this report by the letters A, B, C, D, E, F, and G, respectively.

The additives were used as received and in accordance with the recommendation of the producer except as to the amount of additive used. They varied somewhat in consistency at room temperature but all were liquid when warmed slightly. All were combined directly with the bituminous material but in one case the treated material was used in combination with quicklime.

The objectives of this study did not include the development of an additive and consequently additives were not analyzed to determine their composition. However, it was considered important to determine to what extent an additive affects the characteristics of the bituminous materials, especially those characteristics that are limited by specifications. Combinations of additives and bituminous materials, therefore, were analyzed and such analyses are included herein.

EFFECT OF ADDITIVES ON THE RETENTION OF BITUMINOUS COATINGS IN THE PRESENCE OF MOISTURE: (*A*) WITH 2 PERCENT OF ADDITIVE, COMPLETE COATING WAS RETAINED; (*B*) 1 PERCENT OF ADDITIVE WAS ONLY PARTIALLY SATISFACTORY; AND (*C*) WITHOUT AN ADDITIVE THE BITUMINOUS COATING WAS ALMOST COMPLETELY STRIPPED.

The bituminous materials used were MC–2 cut-back asphalt, RC–2 cut-back asphalt, RT–5 road tar, and asphalt cement of 120 to 150 penetration.

Analyses of the three liquid materials with and without additives are given in tables 1, 2, and 3. Corresponding analyses of the asphalt cement were not made. It will be observed from those tables that the effect of additives upon the original characteristics of the bituminous materials, in general, is too slight to be of importance unless the characteristics of the untreated material are at or near the specification limits.

The results of the thin-film oven test, which is an accelerated weathering test, show that no detrimental effects attributable to the additive should be expected to develop as a result of oxidation in service. For all practical purposes, therefore, the chemical and physical effects of the additives on the acceptability or serviceability of the bituminous material with which they are used, may be disregarded.

The aggegates used in the investigation vary in their hydrophilic properties and include materials having a record of unsatisfactory service behavior with respect to film stripping. The following materials were used: Two gravels, designated herein as No. 1 and No. 2 respectively; two granites designated as No. 1 and No. 2, one quartzite, one limestone, one limerock, one trap rock. The physical properties of these aggregates are given in table 4.

It will be observed, by reference to the tables, that not all of the various combinations of additives and bituminous materials were used with all aggregates. However, it is believed that the data obtained are sufficient to provide the information sought in the investigation.

THREE DIFFERENT TESTS MADE ON MIXTURES

The procedure adopted for examining and comparing the different additives was to prepare bituminous mixtures with and without additives and to subject them to the action of water. The tests used were as follows:

The modified Oberbach test.—This is a static immersion test which was performed under the following conditions

TABLE 1.—*Analyses of MC–2 asphalt with and without additives* [1]

	Untreated asphalt, No. 62533	Identification and amount of additive in bituminous material (No. 62533)									Untreated asphalt, No. 64439	Identification and amount of additive in bituminous material (No. 64439)					
		A			B			C			G	D			E		F
		1 percent	2 percent	4 percent	1 percent	2 percent	4 percent	0.5 percent	1 percent	2 percent	0.65 percent		0.5 percent	1 percent	2 percent	2 percent	4 percent	2 percent
Specific gravity 77°/77° F	0.966	0.967	0.967	0.968	0.967	0.967	0.967	0.967	0.966	0.964	0.966	0.962	0.962	0.961	0.961	0.962	0.963	0.962
Flash point, °F	135	-----	-----	140	-----	-----	140	140	145	155	145	150	145	145	140	145	140	145
Furol viscosity at 140° F., seconds	145.9	142.0	140.3	136.3	143.8	141.0	140.7	145.0	135.5	122.2	132.6	163.0	159.8	157.8	150.0	152.2	144.0	153.8
Distillation:																		
Distillate percentage by volume of total distillate to 680° F.:																		
To 374° F	Trace	Trace	2.0	Trace	2.0	2.0	2.0	2.0	2.0	3.9	2.0	3.4	2.1	2.1	2.1	2.0	2.0	2.0
To 437° F	8.2	8.0	9.8	8.2	10.0	6.0	8.2	8.0	11.8	9.8	12.0	8.5	10.5	12.5	10.6	10.2	10.0	14.0
To 500° F	44.9	44.0	43.1	44.9	44.0	44.0	40.8	44.0	45.1	45.1	46.0	44.7	44.1	45.8	44.7	44.9	44.0	46.0
To 600° F	81.6	80.0	76.5	79.6	80.0	78.0	77.6	78.0	80.4	82.4	82.0	80.8	79.8	81.3	78.7	81.6	82.0	80.0
Residue, percentage by volume total sample less distillate recovered	75.0	75.0	74.5	75.5	75.0	75.0	75.5	75.0	74.5	74.5	75.0	76.5	76.2	76.0	76.5	75.5	75.0	75.0
Tests on residue:																		
Penetration at 77° F. 100 gm. 5 seconds	194	197	194	224	238	240	330	196	188	228	215	182	195	207	250	171	182	190
Ductility at 77° F. 5 centimeters per minute, centimeters	122	113	163	182	111	110	79	113	119	111	144	140	160	205	215	160	180	158
Total soluble in CCl_4, percent	99.92	99.89	99.86	99.81	99.92	99.91	99.92	99.96	99.95	99.94	99.94	99.87	99.93	99.96	99.96	99.94	99.96	99.94
Organic insoluble, percent	.05	.07	.05	.05	.04	.04	.04	.02	.03	.05	.06	.09	.06	.04	.04	.05	.04	.05
Inorganic insoluble, percent	.03	.04	.09	.14	.04	.05	.04	.02	.02	.01	0	.04	.01	0	0	.01	0	.01
Oliensis spot test	Neg.	-----	-----	Neg.	-----	-----	Neg.	-----	-----	-----	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.	Neg
Thin film oven tests at 325° F. of 50 milliliter samples:																		
5-hour test:																		
Loss in weight, percent	24.8	24.3	24.2	23.4	24.4	24.4	23.7	24.5	24.9	25.0	25.1	23.4	23.4	23.3	22.7	23.5	23.6	23.6
Residue penetration, 77° F., 100 gm., 5 seconds	65	63	64	77	72	78	102	74	81	97	74	58	68	74	107	70	78	80
Residue ductility, 77° F., 5 centimeters per minute, centimeters	206	214	203	200	238	228	233	183	140	176	231	240	208	50+	250+	137	215	203
10-hour test:																		
Loss in weight, percent	25.7	25.4	25.3	25.2	25.6	25.7	24.6	25.4	25.8	26.0	25.7	24.1	23.8	23.9	23.4	24.4	24.4	24.2
Residue penetration, 77° F., 100 gm., 5 seconds	39	39	39	43	41	43	47	44	48	56	41	37	42	44	58	40	46	46
Residue ductility, 77° F., 5 centimeters per minute, centimeters	78	78	64	46	110	123	110	95	137	162	95	78	140	165	218	45	90	100
15-hour test:																		
Loss in weight, percent	26.0	25.7	25.6	25.4	25.9	25.8	25.5	26.0	26.2	26.3	26.4	24.5	24.3	24.3	24.2	24.8	25.0	24.7
Residue, penetration at 77° F., 100gm, 5 seconds	30	28	27	27	31	32	32	32	34	41	31	28	29	30	38	31	34	32
Residue, ductility at 77° F., 5 centimeters per minute, centimeters	22	15	11	10	15	21	21	27	53	96	27	12	16	22	78	16	17	26

[1] Bituminous material No. 62533 was used in the stripping tests of all aggregates and in the compression tests of all aggregates excepting gravel No. 1 and granite No. 2 in which bituminous material No. 64439 was substituted.

TABLE 2.—*Analyses of RC–2 asphalt with and without additives*

	Untreated asphalt	Identification and amount of additive in bituminous material								
		B		C		D		E		F	G
		1 percent	2 percent	0.5 percent	1 percent	0.5 percent	2 percent	2 percent	4 percent	2 percent	0.65 percent
Specific gravity, 77°/77° F	0.962	0.963	0.963	0.961	0.960	0.963	0.962	0.962	0.962	0.962	0.962
Flash point, °F	90	-----	95	-----	95	95	90	95	95	90	95
Furol viscosity at 140° F., seconds	229.8	217.2	214.4	222.0	205.8	225.0	199.9	217.4	201.7	219.4	220.8
Distillation:											
Distillate, percentage by volume of total distillate to 680° F.:											
To 374° F	43.8	39.6	40.4	44.7	42.6	42.6	41.7	37.5	40.0	45.8	41.7
To 437° F	70.8	68.8	68.1	70.2	70.2	70.2	72.9	66.7	64.0	70.8	70.8
To 500° F	83.3	83.3	83.0	85.1	83.0	85.1	83.3	83.3	80.0	83.3	83.3
To 600° F	93.8	91.7	91.5	93.6	91.5	93.6	91.7	91.7	95.8	91.7	93.8
Residue, percentage by volume, total sample less distillate recovered	76.0	76.0	76.5	76.5	76.5	76.5	76.0	76.0	75.0	76.0	76.0
Tests on residue:											
Penetration, 100 gm., 77° F., 5 seconds	81	90	102	85	95	89	109	83	87	91	92
Ductility, 77° F., 5 centimeters per min., centimeters	250+	194	218	218	224	220	250+	225	195	250+	250+
Total soluble in CCl_4, percent	99.93	-----	99.94	-----	99.97	99.96	99.92	99.97	99.95	99.96	99.96
Organic insoluble, percent	.05	-----	.04	-----	.03	.04	.08	.03	.05	.04	.03
Inorganic insoluble, percent	.02	-----	.02	-----	0	0	0	0	0	0	.01
Thin film oven tests at 325° F. of 50 milliliter samples, 5-hour test:											
Loss in weight, percent	20.8	20.3	20.5	20.9	20.6	20.8	20.4	21.0	21.1	20.7	20.7
Residue, penetration, 77° F., 100 gm., 5 seconds	37	41	46	44	48	46	58	40	45	45	45
Residue, ductility, 77° F., centimeters	128	197	250+	160	150	250+	235	108	125	150	217

TABLE 3.—*Analyses of RT–5 tar with and without additives*

	Untreated	Identification and amount of additive in bituminous material										
		A			B			C			D	G
		1 percent	2 percent	4 percent	1 percent	2 percent	4 percent	0.5 percent	1 percent	2 percent	0.5 percent	0.65 percent
Specific gravity 25°/25° C	1.174	1.173	1.172	1.166	1.172	1.170	1.165	1.172	1.170	1.166	1.168	1.172
Specific viscosity, Engler, at 50° C	17.5	18.1	18.5	18.8	17.9	17.9	17.8	16.3	14.8	13.6	17.1	16.5
Bitumen soluble in CS_2, percent	89.42	89.07	89.34	89.51	89.21	89.45	90.03	89.22	89.03	89.21	88.95	88.67
Organic insoluble, percent	10.53	10.86	10.58	10.40	10.75	10.43	9.93	10.77	10.95	10.76	10.97	11.31
Inorganic insoluble, percent	.05	.07	.08	.09	.04	.12	.04	01	.02	.03	.08	02
Distillation:												
Percentage by weight:												
To 170° C	Trace	Trace	Trace	Trace	Trace	Trace	Trace	.16	.30	0.41	Trace	Trace
170–235° C	3.68	4.04	3.14	4.01	3.20	4.04	2.23	3.49	3.96	4.33	3.67	3.61
235–270° C	10.20	9.22	10.44	10.08	9.55	10.04	8.57	9.08	9.12	9.57	8.68	10.11
270–300° C	9.03	8.96	8.56	9.71	9.06	8.51	12.05	9.03	9.44	9.13	9.09	8.60
Residue	76.13	77.37	76.95	75.60	78.00	76.90	76.80	77.78	77.03	76.14	78.20	77.20
Softening point of residue, ° C	34.8	34.4	34.7	36.3	34.1	34.4	34.5	32.4	33.0	33.1	33.5	34.6
Sulfonation index (0–300° C. fraction)	1.79	----	1.72	1.83	----	1.70	1.64	----	----	1.61	1.53	1.59
Thin film oven test, 50 milliliters, 163° C., 4 hours:												
Loss in weight, percent	28.6	27.8	26.5	25.7	28.3	27.6	27.2	28.0	28.3	28.4	28.6	28.3
Penetration at 25° C., 100 gm. 5 seconds	8	10	14	22	10	15	26	12	12	19	10	12
Ductility at 25° C., 5 centimeters per minute, centimeters	28	206	158	150	194	206	221	250+	250+	233	136	198

A sample of approximately 25 grams was immersed in distilled water at 70° F. immediately after mixing and, after a period of 24 hours immersion, was examined visually to determine the extent of stripping that had occurred.

A sample of approximately 35 grams was cured in air at 70° F. for 24 hours, then immersed in distilled water at 100° F. and, after a period of 24 hours immersion, was examined to determine the extent of stripping.

The results of the above tests are reported in accompanying tables as the estimated area that remained coated at the conclusion of the test.

The modified Nicholson test.—In this test a sample of approximately 100 grams of the mixture was cured in an oven at 140° F. for 24 hours, after which approximately 50 grams was placed in a flask partially filled with distilled water at 77° F. The stoppered flask was placed in a frame which rotates in a water bath at 77° F. After 30 minutes of rotation, the temperature of the water bath was raised to 100° F. and the rotation continued for an additional 15 minutes.

The appearance of the mixture at the end of 45 minutes of rotation is reported in terms of the estimated area that remained coated at the conclusion of the test.

Immersion-compression test.—Tests were made on cylinders of graded mixtures as described in the preceding article in this magazine.

Some specimens were tested immediately after curing and duplicate specimens were tested after immersion in water at room temperature for 4 days. Additional specimens from some of the mixtures were tested after having been subjected to various periods of alternate wetting and drying.

PREPARATION OF MIXTURES FOR COATING AND STRIPPING TESTS

Prior to preparing the mixtures, the aggregates were screened and the fraction passing the 3/8-inch sieve and retained on the No. 4 sieve was used. The dry weight of aggregate used for all mixtures was 600 grams. Aggregates used wet were placed in distilled water and allowed to soak for 24 hours prior to mixing. Before placing in the mixing bowl the wet aggregates were drained until they contained approximately 3.5 percent of free water, due allowance being made for absorption.

TABLE 4.—*Physical analyses of aggregates*

Type of aggregate	Description	Apparent specific gravity	Bulk specific gravity	Absorption	Los Angeles, percentage of wear
				Percent	*Percent*
Gravel No. 1	Gravel containing angular and subangular pieces of quartz, sandstone, quartzite, and chert.	2.64	2.53	1.7	42.8
Gravel No. 2	Gravel containing subangular pieces of quartz, quartzite, gneiss, granite, and schist.	2.69	2.63	.8	33.0
Granite No. 1	Crushed aplitic granite	2.67	2.61	.8	24.7
Granite No. 2	Crushed granite	2.76	2.72	.6	32.5
Quartzite	Crushed feldspathic quartzite.	2.63	2.60	.5	22.8
Limestone	Crushed argillaceous limestone.	2.75	2.72	.4	16.0
Limerock	Crushed limerock having a powdery surface.	2.58	2.37	3.4	41.3
Trap rock	Crushed trap rock (altered diabase).	2.95	2.86	1.1	----

APPARATUS USED IN PERFORMING THE MODIFIED NICHOLSON TEST FOR STRIPPING.

STORAGE TANK FOR SPECIMENS SUBJECTED TO ALTERNATE WETTING AND DRYING. THE TANK WAS FILLED AND DRAINED AS NECESSARY TO PRODUCE A WET OR DRY CONDITION.

Aggregates used wet or dry with liquid bituminous materials were at room temperature. The tar and liquid asphalts were heated to 120° and 175° F. respectively. In preparing the mixtures containing asphalt cement, both aggregate and bitumen were heated to 275° F. prior to mixing.

The amounts of additives used ranged from 0.5 to 4.0 percent of the weight of the bituminous material. This range included the particular percentages recommended by the individual producers.

The bitumen contents used were such that for all mixtures the film thickness on the aggregate particles was theoretically constant. The actual percentage of bitumen, including additives, ranged from 4.6 to 7.6 percent of the dry weight of the aggregate.

In the mixtures containing the additive that required quicklime, the proportion by weight was four parts of quicklime to one part of additive. In preparing the mixtures with wet aggregate, the lime was mixed with aggregate before applying the bituminous material. In preparing the dry mixes lime was applied 15 seconds after the bituminous material had been mixed with the aggregate. The additive was added to the bituminous material prior to the mixing operation.

A kitchen-type mixer was used in preparing the mixtures. This mixer was equipped with a special paddle composed of flexible mixing blades to provide proper mixing with the minimum of crushing. The operating speed was approximately 60 revolutions per minute and the mixing operation was continued until no further increase in coating was obtained or until a tendency to strip was observed.

At the completion of each mixing operation the following samples were taken:

(1) Approximately 25 grams for the immediate static immersion stripping test at 77° F.

(2) Approximately 35 grams for 24 hours of air curing at 70° F. to be followed by the static immersion stripping test at 100° F.

(3) Approximately 100 grams for 24 hours of oven curing at 140° F. to be followed by the modified Nicholson stripping test.

ALL ADDITIVES PRODUCED IMPROVEMENT IN COATING OF WET AGGREGATES

All the bituminous materials, both with and without additives, readily coated the aggregates used in this investigation when the aggregates were dry.

When the aggregates were wetted prior to mixing with the bituminous materials and in the manner

TABLE 5.— *Results of coating tests using wet aggregates. Estimated percentage of the area well coated*

MIXTURES WITH MC-2 MATERIAL

Type of aggregate	Untreated material	Identification and percentage of additive in the bituminous material															
		A			B			C			D			E		F	G
		1	2	4	1	2	4	0.5	1	2	0.5	1	2	2	4	2	0.65
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Gravel No. 1	50	100	100	100	80	100	100	100	100	100	100	------	100	50	75	100	100
Gravel No. 2	50	100	100	100	80	95	100	100	100	100	100	------	------	------	------	------	100
Granite No. 1	65	95	95	100	60	80	95	90	100	100	95	------	------	------	------	------	100
Granite No. 2	65	95	100	100	90	100	100	100	100	100	95	------	100	70	95	100	100
Quartzite	40	90	95	100	50	100	100	100	100	100	95	100	100	40	85	100	100
Limestone	80	100	100	100	95	100	100	95	95	100	100	100	100	------	------	------	100
Limerock	50	------	------	------	------	100	100	------	------	------	------	------	100	90	100	100	------
Trap rock	80	------	------	------	85	------	------	------	------	------	------	------	------	------	------	------	95
Average	60	97	98	100	77	96	99	98	99	100	98	100	100	63	89	100	99
MIXTURES WITH RT-5 MATERIAL																	
Gravel No. 1	90	100	100	100	80	80	85	95	95	100	------	------	------	------	------	------	100
Gravel No. 2	95	100	100	100	85	85	90	95	95	100	------	------	------	------	------	------	100
Granite No. 1	95	100	100	100	85	95	95	95	95	100	95	------	------	------	------	------	100
Granite No. 2	95	100	100	100	90	90	85	100	100	100	------	------	------	------	------	------	100
Quartzite	80	100	100	100	65	65	85	85	85	85	100	------	------	------	------	------	100
Limestone	95	100	100	100	85	85	95	60	60	[1] 70	95	------	------	------	------	------	100
Average	92	100	100	100	82	83	89	88	88	93	97	------	------	------	------	------	100
MIXTURES WITH RC-2 MATERIAL																	
Gravel No. 1	40	------	------	------	85	95	------	100	100	------	------	------	100	30	50	100	100
Granite No. 1	40	------	------	------	85	90	------	100	100	------	95	------	------	------	------	------	100
Quartzite	20	------	------	------	90	100	------	100	100	------	95	------	100	20	50	100	100
Limestone	50	------	------	------	95	100	------	90	100	------	95	------	------	------	------	------	100
Limerock	10	------	------	------	------	90	100	------	------	------	------	------	100	50	95	100	------
Average	32	------	------	------	89	95	100	98	100	------	95	------	100	33	65	100	100

[1] In a test in which the bituminous material contained 4 percent of the additive, the estimated area well coated was 95 percent.

previously described, the coating obtained varied with the amount and kind of additives used. The results are shown in table 5 and are reported as the areas that upon visual inspection appeared to be well coated. Areas that appeared to be only slightly colored by the bituminous material were considered as uncoated. It will be observed that without an additive, the coating obtained with MC–2 asphalt varied from 40 to 80 percent and averaged 60 percent. With RC–2 asphalt the coating varied from 10 to 50 percent and averaged 32 percent, while with RT–5 tar it ranged from 80 to 95 and averaged 92 percent.

When additives were used with MC–2 and RC–2 asphalt a definite improvement in coating was always

TABLE 6.— *Results of static immersion tests at 70° F. on uncured mixtures using wet aggregates. Estimated percentage of area remaining coated*

MIXTURES WITH MC–2 MATERIAL

Type of aggregate	Untreated material	Identification and percentage of additive in the bituminous material															
		A			B			C			D			E		F	G
		1	2	4	1	2	4	0.5	1	2	0.5	1	2	2	4	2	0.65
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Gravel No. 1	0	10	20	80	30	60	95	90	95+	100	50		100	0	25	100	85
Gravel No. 2	0	15	70	90	40	95	95+	95	95+	100	85						90
Granite No. 1	0	10	15	40	10	30	80	30	70	95	40						50
Granite No. 2	0	15	70	90	20	80	95	100	100	100	90		100	5	25	100	95
Quartzite	0	20	70	85	20	70	90	60	100	100	15	100	100	0	15	100	80
Limestone	5	85	80	95	50	80	90	10	20	[1] 50	50	100	100				80
Limerock	0					90	95						[2] 95	20	70	[1] 50	
Trap rock	0				5												5
MIXTURES WITH RT–5 MATERIAL																	
Gravel No. 1	10	50	75	95	20	40	80	70	85	95							95
Gravel No. 2	10	75	80	95+	30	80	90	85	95	95+							95
Granite No. 1	0	15	50	70	20	50	95	80	90	95+	85						95+
Granite No. 2	10	70	60	85	75	85	90	90	95	95							95+
Quartzite	10	70	85	90	20	75	85	70	80	90	95+						100
Limestone	0	70	85	80	15	20	70	5	10	[3] 15	80						90
MIXTURES WITH RC–2 MATERIAL																	
Gravel No. 1	5				75	95		95	95				100	0	5	100	95
Granite No. 1	5				20	35		80	85		85						90
Quartzite	0				40	85		90	90		90		95+	5	15	95+	95
Limestone	5				60	85		50	75		80						95
Limerock	0					85	95+						[2] 80	20	75	[2] 80	

[1] In a test in which the bituminous material contained 4 percent of the additive, the estimated area remaining coated was 95 percent.
[2] In a test in which the bituminous material contained 4 percent of the additive, the estimated area remaining coated was 100 percent
[3] In a test in which the bituminous material contained 4 percent of the additive, the estimated area remaining coated was 75 percent.

TABLE 7.— *Results of static immersion tests at 100° F. on air-cured mixtures using wet aggregates. Estimated percentage of area remaining coated*

MIXTURES WITH MC–2 MATERIAL

Type of aggregate	Untreated material	Identification and percentage of additive in the bituminous material															
		A			B			C			D			E		F	G
		1	2	4	1	2	4	0.5	1	2	0.5	1	2	2	4	2	0.65
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Gravel No. 1	10	5	15	85	95	100	100	95+	95	100	60		100	70	85	100	100
Gravel No. 2	20	15	85	100	95+	100	100	100	100	100	100						100
Granite No. 1	5	5	10	95	75	95+	100	100	100	100	85						95+
Granite No. 2	10	10	30	100	95+	100	100	100	100	100	85		100	20	75	100	100
Quartzite	5	5	15	100	90	100	100	85	100	100	10	80	100	10	85	100	90
Limestone	10	95	95+	100	95	95+	100	90	90	95+	95	100	100				95+
Limerock	20					100	100						100	95	100	95+	
Trap rock	5				50												80
MIXTURES WITH RT–5 MATERIAL																	
Gravel No. 1	100	100	100	100	95	95	95	95+	100	100							100
Gravel No. 2	95	100	95+	100	90	95+	100	100	100	100							100
Granite No. 1	95	100	95	100	95	100	100	100	100	100	100						100
Granite No. 2	100	100	100	100	95+	95+	100	100	100	100							100
Quartzite	100	95+	100	100	95	95+	100	95+	100	100	100						100
Limestone	30	100	100	100	80	95	100	40	50	70	95+						100
MIXTURES WITH RC–2 MATERIAL																	
Gravel No. 1	10				90	95+		95+	100				100	10	65	100	100
Granite No. 1	5				70	95		100	100		95+						95+
Quartzite	10				90	95		100	100		90		100	10	10	100	100
Limestone	40				90	95+		95	95		95						95+
Limerock	40					90	100						95+	80	90	95+	

MIXTURES FOR THE COATING AND STRIPPING TESTS WERE PREPARED IN A KITCHEN-TYPE MIXER.

obtained with some percentage of additive but the optimum percentage of a given additive was not the same for all aggregates.

When the additives were used with tar, the benefit obtained was less apparent as the untreated tar coated the aggregate reasonably well. Of the five additives used with the tar, 100 percent coating of all aggregates was obtained with 1 percent of additive A and with 0.65 percent of additive G. Approximately the same result was obtained with 0.5 percent of additive D on the three aggregates used with tar. No improvement was afforded by the use of as much as 4 percent of additive B with tar. Use of additive C with tar resulted in complete coating of four of six aggregates with 2 percent of additive. With one aggregate practically the same result was obtained with 0.5, 1, and 2 percent of additive C as with the tar alone. With the remaining aggregate, the tar alone was more satisfactory than when 2 percent of additive C was used, and 4 percent of this additive produced no improvement over the tar without additive.

RESULTS OF STRIPPING TEST ON MIXTURES MADE WITH WET AGGREGATE DISCUSSED

The effect of the additives in improving the ability of the three liquid bituminous materials to adhere to wet aggregate after being subjected to the action of water in the static immersion test is shown in tables 6 and 7.

Table 6 gives the results of the static immersion test on the wet, uncured mixtures. The conditions under which this test is made are admittedly severe as is evidenced by the fact that at its conclusion a maximum of 10 percent of area remained coated when additives were not used. However, when additives were used the coating retained was substantially increased although, in the case of additive E, this increase was substantial with one aggregate only.

The improvement obtained with additives varied considerably. It varied with the kind and percentage of additive, with the type of bituminous material, and with the type of aggregate. In practically every instance, increasing the amount of additive resulted in increased resistance to stripping but the rate of improvement varied considerably for different additives. Also, the different additives varied considerably in their effectiveness with different aggregates.

Table 7 gives the results of the static immersion test made at 100° F. on the mixtures that contained wet aggregate and which were air-cured at 70° F. for 24 hours before testing. It will be observed that when additives were not used, the asphaltic materials did not provide a coating resistant to stripping but that the tar did except when used with limestone.

In general, when additives were used, definite improvement was obtained, although the improvement in the tar mixtures was limited to the limestone since the other tar-coated aggregates showed good resistance to stripping without an additive. The amount of improvement resulting from the use of the additives, as measured by this test, varied as in the tests of the uncured mixtures.

The tests with the untreated materials showed that the resistance to stripping had been greatly improved by the curing prior to the test and it appears that the benefits obtained by curing, in general, more than compensate for the detrimental effect of the higher test temperatures. An exception was found when 2 percent or less of additive A was used with MC–2 asphalt. With such materials some aggregates gave more satisfactory results when the mixtures were tested immediately after mixing.

RESULTS OF STRIPPING TESTS ON MIXTURES MADE WITH DRY AGGREGATES

Tables 8, 9, and 10 give the results obtained on the mixtures prepared with dry aggregates. Tables 8 and 9 show the results of the static immersion tests on the uncured and on the air-cured samples, respectively, and table 10 gives the results of the modified Nicholson tests.

Table 8 shows that when mixtures containing no additives were tested without curing, the amount of coating retained varied considerably, not only with the aggregate used but also with the kind of bituminous material. With MC–2 asphalt, the maximum coating retained was 50 percent; with RC–2 asphalt, the maximum was 85 percent; with tar, an excellent coating was retained on all aggregates except the limestone. It is interesting to note that in the untreated MC–2 and RC–2 mixtures, this aggregate showed a better resistance to stripping than any of the others, while in the tar mixtures it showed the poorest resistance of any of the aggregates. The actual amount of coating retained was greater with RC–2 asphalt than with tar.

When additives were used, considerable improvement resulted in most cases. Practically, complete coating was retained with all percentages of additives B, C,

TABLE 8.—*Results of static immersion tests at 70° F. on uncured mixtures using dry aggregates. Estimated percentage of area remaining coated*

MIXTURES WITH MC-2 MATERIAL

Aggregate	Untreated material	Identification and percentage of additive in the bituminous material								
		A			B		C		D	G
		1	2	4	1	2	0.5	1	0.5	0.65
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Gravel No. 1	25	20	70	90	100	100	95	95+	95	100
Gravel No. 2	50	60	90	95+	100	100	100	100	100	100
Granite No. 1	30	30	40	70	100	100	100	100	100	100
Granite No. 2	10	20	15	50	100	100	100	100	90	100
Quartzite	15	30	60	75	95	100	95+	100	90	95+
Limestone	50	95	95	100	100	100	95+	100	100	100
Trap rock	10				100					100

MIXTURES WITH RT-5 MATERIAL

Gravel No. 1	100	95	90	95+	100	95+	100	100		100
Gravel No. 2	100	100	100	100	100	100	100	100		100
Granite No. 1	100	85	80	85	100	100	100	100	100	100
Granite No. 2	95+	50	30	20	100	100	100	100		100
Quartzite	100	95	85	75	90	100	95+	100	100	100
Limestone	70	100	100	100	90	95+	70	80	100	100

MIXTURES WITH RC-2 MATERIAL

Gravel No. 1	75				100	100	100	95+		95+
Granite No. 1	25				100	100	100	100	100	95+
Quartzite	60				95	100	95	95+	95–	95+
Limestone	85				95+	100	95	100	100	95+

TABLE 9.—*Results of static immersion tests at 100° F. on air-cured mixtures using dry aggregates. Estimated percentage of area remaining coated*

MIXTURES WITH MC-2 MATERIAL

Aggregate	Untreated material	Identification and percentage of additive in the bituminous material								
		A			B		C		D	G
		1	2	4	1	2	0.5	1	0.5	0.65
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Gravel No. 1	25	20	90	95+	100	100	80	95	95+	100
Gravel No. 2	30	95	100	100	100	100	100	100	100	100
Granite No. 1	15	40	95	95	95+	100	100	100	95+	95+
Granite No. 2	10	30	65	75	100	100	100	100	95	100
Quartzite	15	20	90	80	95+	100	85	100	50	90
Limestone	30	95	100	100	100	100	90	100	95+	95+
Trap rock	15				100					100

MIXTURES WITH RT-5 MATERIAL

Gravel No. 1	100	100	95+	100	100	100	100	100		100
Gravel No. 2	100	100	100	100	100	100	100	100		100
Granite No. 1	100	95	100	95	100	100	100	100	100	100
Granite No. 2	100	50	70	30	100	100	100	100		100
Quartzite	100	95	95	90	100	100	100	100	95+	100
Limestone	75	100	100	100	95	100	80	80	95+	95+

MIXTURES WITH RC-2 MATERIAL

Gravel No. 1	85				100	100	100	100		100
Granite No. 1	50				100	100	100	100	100	100
Quartzite	40				100	100	100	100	100	100
Limestone	90				100	100	95+	95+	100	100

D, and G, except that tar with additive C showed only slight improvement over tar alone when used with limestone.

Additive A was not used with RC–2 asphalt. When used with MC–2 asphalt, some improvement resulted, which generally increased with the amount of additive, but results varied considerably with different aggregates. When additive A was used with tar, no improvement resulted except in the case of the limestone. When the gravels were used, the results with and without additive A in tar were substantially the same. With the granites and quartzite, the untreated tar was more satisfactory than the tar containing additive A and it will be noted that with two of these aggregates, that is, with granite and quartzite, greater amounts of additive A in tar were less satisfactory than small amounts.

The results of the tests of mixtures containing dry aggregates which were air-cured before testing are given in table 9. In the tests without additives the

TABLE 10.—*Results of modified Nicholson stripping tests on oven-cured mixtures using dry aggregates. Estimated percentage of area remaining coated at the end of the 100° F. test period*

Aggregate	Untreated material	Identification and percentage of additive in the bituminous material								
		A			B		C		D	G
		1	2	4	1	2	0.5	1	0.5	0.65
MIXTURES WITH MC-2 MATERIAL										
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Gravel No. 1	90	95	90	100	95	100	95	100	95	90
Gravel No. 2	90	100	100	100	85	100	100	100	95	100
Granite No. 1	60	95	95	95	95	100	100	100	95+	100
Granite No. 2	80	95	100	100	100	100	100	100	100	100
Quartzite	75	95	90	90	100	100	100	95	100	100
Limestone	100	100	100	100	100	100	100	95+	100	100
Trap rock	80				95					100
MIXTURES WITH RT-5 MATERIAL										
Gravel[2] No. 1	100	100	100	100	95	90	100	100		100
Gravel No. 2	100	100	100	100	95	100	100	100		100
Granite No. 1	95	100	100	100	100	100	100	100		100
Granite No. 2	100	100	100	95	100	100	100	100		100
Quartzite	100	100	100	100	95	100	100	100		100
Limestone	100	100	100	100	100	100	95	100		100
MIXTURES WITH RC-2 MATERIAL										
Gravel No. 1	90				100	95	95	100		100
Granite No. 1	85				95	100	100	100		100
Quartzite	95				95	95	90	100		100
Limestone	100				100	100	100	100		100

results were similar to those shown in table 8 in that the coating retained varied considerably. They were least satisfactory with MC–2 asphalt, more satisfactory with RC–2 asphalt, and complete coating with tar was retained except on the limestone

When additives were used, the results were also similar to those with the uncured mixtures shown in table 8 in that practically complete coating was retained with all percentages of additives B, C, D, and G, and that additive C was only slightly beneficial when used in tar-limestone mixes. Additive A was beneficial with MC–2 asphalt in varying degrees. It was beneficial with tar only when used with limestone. With granite No. 2 and tar it was less satisfactory than tar alone and the results obtained were still less satisfactory when the amount of additive A was increased from 2 to 4 percent.

The results of the tests made by the modified Nicholson method on oven cured mixtures using dry aggregates are given in table 10. From these results it will be observed that with several aggregates some improvement resulted from using additives with MC–2 asphalt. However, the conditions of the test were not sufficiently severe to demonstrate differences between the additives or the effect of using different percentages of additive. The effect of agitation at the testing temperature of 100° F. was apparently more than compensated for by the 24 hours of oven curing at 140° F.

This study was confined mainly to the use of liquid materials in order to compare results obtained when unheated aggregates, wet or dry, were used. However, since the opinion has been frequently expressed that a coating of asphalt cement on aggregate is generally resistant to moisture, a limited number of tests with asphalt cement were included in this investigation in order to obtain comparative data on the subject.

An asphalt cement of 120–150 penetration, with and without additives B and C, was used to coat gravel No. 1, granite No. 1, quartzite, and limestone. In preparing these mixtures the aggregates were heated to 275° F. All aggregates coated readily whether or not additives were used.

These mixtures were subjected to the three stripping tests. At least 90 percent of the coating was retained in every instance and in 16 of 24 tests 100 percent was retained. For all practical purposes it can be stated that the asphalt cement effectively coated the four aggregates used and the coating was not affected in the tests applied. The test data are not included in the tables and they do not warrant further discussion.

Considering the results of stripping tests in tables 6, 7, 8, 9, and 10, the following statements appear to be warranted:

1. In practically all cases, additives were of benefit in the retention of liquid asphaltic films on aggregate subjected to the action of water.
2. The benefit thus obtained is increased if the coated aggregate is allowed to cure before being subjected to moisture.
3. Additives appear to be of little benefit when used with tar except when mixtures of tar and wet aggregate are subjected to the action of water before curing.
4. The effectiveness of additives varies both with the bituminous material and with the aggregate used with them.
5. The optimum percentage of a given additive is not the same for all aggregates.

ADDITIVES IMPROVE STABILITY OF DENSE-GRADED MIXTURES SUBJECTED TO MOISTURE

To obtain information on the effect of additives in reducing the loss of compressive strength of dense-graded bituminous mixtures due to absorbed moisture, a number of specimens were made and subjected to the immersion-compression test. These specimens were cylinders 4 by 4 inches except those containing trap rock. These were 3 by 3 inches. The composition of the bituminous mixtures used in all of the specimens was as follows:

TABLE 11.—*Results of immersion-compression test. Each value is the average of values from 3 specimens. MC–2 asphalt used in all mixes*

Aggregate and additive used	Bitumen-aggregate ratio by weight		Volumetric analysis of cured specimens			Compressive strength tests on dry specimens	Tests on specimens immersed in water 4 days			Retained strength [2]
	As mixed	After curing	Aggregate	Bitumen	Air		Compressive strength	Absorption [1]	Volumetric swell	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Lb. per sq in.*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*
GRANITE No. 1, ROUND 1:										
None	6.0	5.6	81.3	12.3	6.4	62	17	3.8	4.6	27
2 percent A	6.0	5.6	82.0	12.5	5.5	61	55	1.4	4	90
1 percent B	6.0	5.6	82.0	12.5	5.5	59	57	1.5	6	97
0.5 percent D	6.0	5.6	81.8	12.4	5.8	62	43	1.9	1.4	69
0.65 percent G	6.0	5.6	82.3	12.6	5.1	65	53	1.6	1.1	82
GRANITE No. 1, ROUND 2:										
None	6.0	5.6	81.8	12.3	5.9	58	10	3.7	5.8	17
2 percent A	6.0	5.6	82.3	12.5	5.2	58	54	1.1	1	93
1 percent B	6.0	5.6	82.4	12.5	5.1	55	54	1.0	.5	98
0.5 percent D	6.0	5.6	82.3	12.5	5.2	52	34	1.7	1.8	65
0.65 percent G	6.0	5.6	82.4	12.4	5.2	58	43	1.6	1.2	74
GRANITE No. 1, ROUND 3:										
None	6.0	5.6	81.6	12.3	6.1	44	6	5.1	8.0	14
2 percent A	6.0	5.6	81.5	12.3	6.2	45	36	1.6	.3	80
1 percent B	6.0	5.6	81.8	12.3	5.9	44	43	1.6	.5	98
0.65 percent G	6.0	5.6	81.6	12.4	6.0	45	27	2.6	2.6	60
LIMESTONE:										
None	6.0	5.6	83.3	12.9	3.8	56	39	1.6	1.7	70
2 percent A	6.0	5.6	83.8	13.0	3.2	49	51	.6	0	104
1 percent B	6.0	5.6	83.6	13.0	3.4	50	56	.8	3	112
0.5 percent D	6.0	5.6	83.5	12.9	3.6	49	53	.8	.1	108
0.65 percent G	6.0	5.6	83.8	13.0	3.2	51	60	.6	1	118
GRAVEL No. 1:										
None	6.0	5.6	81.8	12.3	5.9	43	33	1.6	4	77
2 percent A	6.0	5.6	81.8	12.4	5.8	45	36	1.4	0	80
1 percent B	6.0	5.6	81.9	12.3	5.8	39	39	1.2	0	100
1 percent D	6.0	5.6	81.9	12.3	5.8	38	36	1.5	3	95
GRANITE No. 2:										
None	5.7	5.3	83.1	12.2	4.7	64	35	1.5	1.3	55
2 percent A	5.7	5.3	83.0	12.3	4.7	61	50	1.0	.1	82
1 percent B	5.7	5.3	83.2	12.2	4.6	61	59	1.0	1	97
1 percent D	5.7	5.3	83.0	12.3	4.7	61	57	1.2	5	93
TRAP ROCK:										
None	5.5	5.1	81.2	11.7	7.1	57	0			0
1 percent B	5.5	5.1	81.5	11.7	6.8	60	62	1.5	.3	103
0.65 percent G	5.5	5.1	81.1	11.7	7.2	65	45	2.1	1.0	69

[1] Ratio of moisture to aggregate by weight.
[2] Ratio of compressive strength of the immersed specimens to the compressive strength of the corresponding dry specimens.

Coarse aggregate.—Five of the materials used in the coating and stripping tests graded from 0.5 inch to No. 10.

Fine aggregate.—Potomac River sand passing the No. 10 sieve.

Dust.—Silica dust.

Bitumen.—MC–2 cut-back asphalt without additive and with the minimum amount of additive shown to have been reasonably satisfactory in the stripping test.

The composition, by weight, was as follows:

Aggregate sizes:	*Percent*
0.5 inch to No. 4	47
No. 4 to No. 10	13
No. 10 to No. 200	30
Passing No. 200	10

The bitumen content, in terms of dry aggregate, was 5½ to 6 percent.

The mixtures were prepared in a laboratory twin-pug mixer. Aggregates were dry and at room temperature. Bituminous materials were heated to 175° F. The period of mixing was 1½ minutes and was sufficient to insure uniform coating.

When the mixing operation was completed, the mixture was placed in a large pan and air cured at room temperature for 24 hours before molding. During this period it was stirred occasionally to expedite curing.

The test specimens were molded by direct compression, using the double plunger method, and a load of 3,000 pounds per square inch was maintained for 2 minutes. They were cured in an oven at 140° F. for 24 hours and then allowed to cool overnight to room temperature.

Immediately thereafter one-half of the specimens were tested for compressive strength and the others were immersed in water at approximately 77° F. for 4 days and then, while still wet, were also tested for compressive strength.

To provide information on the permanency of any benefit obtained by the use of additives, an additional series of test specimens was made. The coarse aggregates used were gravel No 1 and granite No. 2. The fine aggregate, filler, and bitumen were the same as those used in the specimens previously described.

Because of the number of specimens required in this series, a slight variation had to be made in their preparation. All the mixtures from which they were formed were made in one day but only one-third of them could be molded or oven cured at one time. A third of the total required was therefore molded on each of three successive days, oven cured for 24 hours immediately after molding, and then stored in air until the entire series had been thus processed. Consequently, the average curing time before molding, as well as the period of air storage after oven curing, was two days instead of one. A test group included specimens representing each of these curing and storage conditions.

Some of the specimens of this series were tested as previously described, that is, some were tested after oven curing and some were immersed in water for 4 days before being tested.

Other specimens of this series were subjected to several cycles of immersion in water and subsequent air curing before being tested. A cycle consisted of

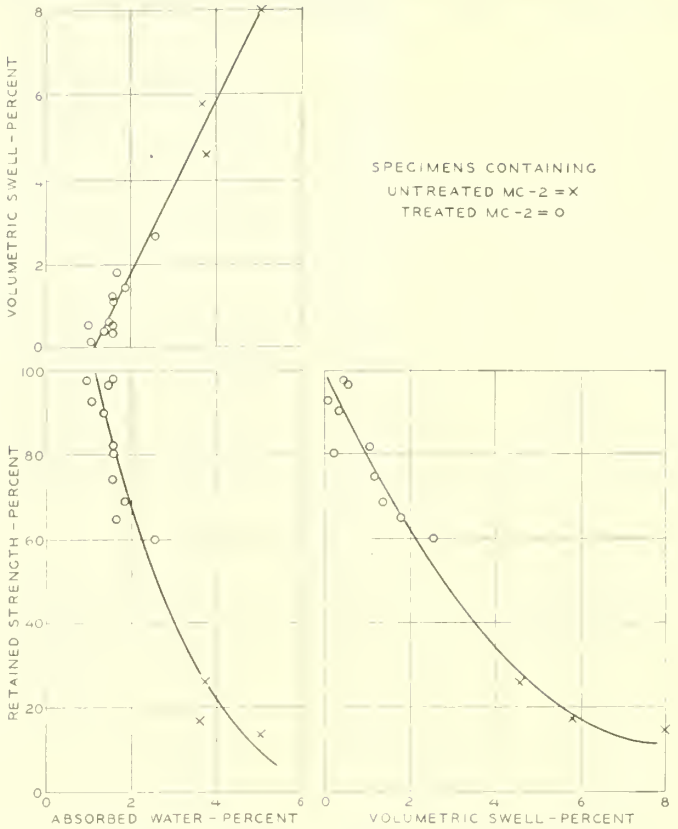


FIGURE 1.—RELATIONS BETWEEN ABSORBED WATER, VOLUMETRIC SWELL, AND RETAINED STRENGTH AFTER 4 DAYS IN WATER; GRANITE NO. 1 MIXTURES.

immersion in water at approximately 77° F. for 4 days, followed by drying in laboratory air for 10 days. Specimens subjected to this method of aging were tested at the end of the immersion period while they were still wet. Duplicate specimens, cured in air, were also tested dry after the same storage period used for the wet specimens. In addition, a number of specimens were buried in soil out of doors with the upper face exposed to the elements. Testing of these specimens will be delayed until they have aged considerably.

The compressive-strength test to which all specimens were subjected was made on the unconfined specimen at a temperature of 77° F. The load was applied to provide vertical deformation at the rate of 0.05 inch per minute per inch of height of specimen. The compressive strength was taken as the maximum unit load recorded.

The results of this test are given in table 11 and show that, in every instance when additives were used, the amount of moisture absorbed was definitely reduced. They reduced absorption from 0.1 to 3.5 percent, the average being 1.6 percent. The volumetric swell resulting from the absorbed moisture was also decreased by the use of additives. The reduction ranged from 0.1 to 7.7 percent and averaged 3.1 percent. In general the swell was approximately proportional to the absorbed moisture.

The specimens not subjected to the action of water varied in compressive strength with different coarse aggregates as might be expected, but there was practically no difference in strength as a result of the use or nonuse of any of the additives.

When the specimens were immersed in water for 4 days, however, the effect of moisture on compressive strength was very pronounced. In every instance, when additives were not used, the compressive strength retained, in terms of the strength of dry specimens, was greatly reduced. It ranged from 0 to 77 percent and averaged 44 percent for the five aggregates used.

When additives were used, the percentage of strength retained in all cases was much greater than when they were not used. It ranged from 60 to 118 percent and averaged 90 percent. The effect of the different additives varied in degree but in every instance they were beneficial. It is interesting to note that the retention of compressive strength, due to the use of additives, expressed as a percentage is approximately equal to the corresponding reduction in percentage of absorbed moisture. As shown in table 12, this relation appears to hold despite the considerable range in density and absorption obtained with the different aggregates.

Figure 1 shows the relation between the absorption, volumetric swell, and retained strength for the treated and untreated mixes containing MC–2 asphalt and granite No. 1. Similar tests of other aggregates were insufficient in number to establish curves and they are therefore not included in this figure. However, it would be expected that these and other aggregates would produce curves of the same general types as those shown.

BENEFITS OBTAINED BY USING ADDITIVES SEEM TO HAVE A CONSIDERABLE DEGREE OF PERMANENCE

The early advantages gained by the use of additives are definitely shown by the results obtained in the 4-day immersion tests. However, these results do not provide information on the long-time retention of such advantages and, to obtain such information, a number

TABLE 12.—*Comparison between absorption and retained strength values for specimens containing MC–2 asphalt after 4 days of immersion*[1]

Coarse aggregate	Water absorbed, average value		Retained strength, average value		Change due to additives		
	Untreated mix	Treated mixes	Untreated mix	Treated mixes	Difference in percentages of water absorbed	Reduction in absorption	Difference in percentages of strength retained
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Granite No. 1 (round 1)	3.8	1.60	27	85	2.20	58	58
Granite No. 1 (round 2)	3.7	1.35	17	83	2.35	64	66
Granite No. 1 (round 3)	5.1	1.93	14	79	3.17	62	65
Limestone	1.6	0.70	70	111	.90	56	41
Gravel No. 1	1.6	1.37	77	92	.23	14	15
Granite No. 2	1.5	1.07	55	91	.43	29	36

[1] Values taken from table 11

of specimens identical with those used in the immersion-compression test were prepared with gravel No. 1 and granite No. 2. Some of these specimens were stored in air for 74 days before testing and others were alternately wetted and dried for the same period before testing. The results of these tests are given in table 13 together with the corresponding test results obtained in the 4-day immersion-compression test. Test results are arranged in four groups, each representing a different storage condition. Groups 1 and 2 represent the storage conditions used in the immersion-compression test and the results given were taken from table 11.

The specimens in group 3 were tested wet after having been alternately wetted and dried for a 74-day period. This storage period began and also ended with a wetting period. The specimens were immersed in water six times and permitted to air dry after each of the first five immersions. The period for each immersion was 4 days and that for each air drying was 10 days. Thus, out of the 74-day storage period, the specimens were in water for a total time of 24 days and in air for a total of 50 days.

The specimens of group 4 were stored for 74 days in laboratory air and then tested dry.

A comparison of the results of group 3 with those of group 2 show that the final water contents for the specimens which were subjected to the 74 days of alternate wetting and drying were higher than those for the specimens which were stored in water for only 4 days. Likewise, the volumetric swells were higher. But increases in absorption and swell due to the five extra immersions did not result in decreased values for compressive strengths. On the contrary, the increases range from 6 to 42 pounds per square inch. Apparently, most of the increase is due to the air curing which occurred during the 50 days of dry storage. This is rather definitely indicated by comparing the results of group 4 with those of group 1 and noting the increased strength which resulted from 74 days of air storage.

Reference to the 4-day strength values shows that all three additives were effective in increasing the compressive strength. The amount of the increase varied with the aggregate and with the additive. The least increase in compressive strength was produced by additives A and D when used with the gravel No. 1, and the greatest amount occurred with additive B in the mix containing the granite No. 2.

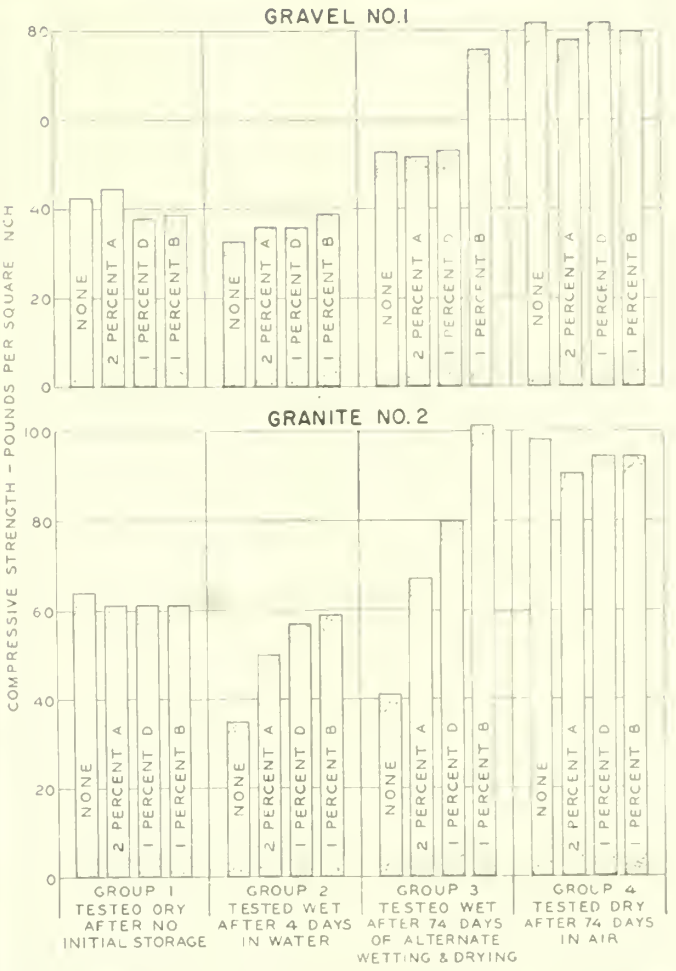

GRAVEL NO.1
GRANITE NO. 2
COMPRESSIVE STRENGTH – POUNDS PER SQUARE INCH
NONE
2 PERCENT A
1 PERCENT D
1 PERCENT B
GROUP 1 TESTED DRY AFTER NO INITIAL STORAGE
GROUP 2 TESTED WET AFTER 4 DAYS IN WATER
GROUP 3 TESTED WET AFTER 74 DAYS OF ALTERNATE WETTING & DRYING
GROUP 4 TESTED DRY AFTER 74 DAYS IN AIR

FIGURE 2.—EFFECT OF ADDITIVES IN IMPROVING THE RETENTION OF STABILITY MIXTURES CONTAINING MC-2 ASPHALT.

The compressive strengths given in table 13 are shown graphically in figure 2. Group 2 of this figure illustrates the relative effect of the three additives in providing higher compressive strengths after the initial 4-day immersion in water. Additives A and D were the least effective and additive B was the most effective. The additives were much more effective when used with granite No. 2 than when used with gravel No. 1.

TABLE 13.—*Comparison of results obtained on specimens subjected to 4 different storage conditions. Each value is the average of values obtained from 3 specimens*

Kind of aggregate and additive in MC-2 material	Compressive strength				Water content [1]			Volumetric swell	
	Group 1, no initial storage, tested dry	Group 2, 4 days in water, tested wet	Group 3, 74 days alternate wetting and drying, tested wet	Group 4, 74 days in air, tested dry	Group 2, after 4 days in water	Group 3, 74 days alternate wetting and drying		Group 2, after 4 days in water	Group 3, after 74 days alternate wetting and drying
						Prior to final 4 days in water	After final 4 days in water		
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
GRAVEL NO. 1:									
None	43	33	53	82	1.6	0.6	2.0	0.4	0.7
2 percent A	45	36	52	78	1.4	.7	1.7	0	[illegible]
1 percent D	38	36	53	82	1.5	.7	2.3	.3	1.2
1 percent B	39	39	76	80	1.2	.8	1.7	0	[illegible]
GRANITE NO. 2:									
None	64	35	41	98	1.5	.3	2.5	1.3	2.9
2 percent A	61	50	67	91	1.0	.5	1.3	1	[illegible]
1 percent D	61	57	80	95	1.2	.4	1.6	.5	1.[illegible]
1 percent B	61	59	101	95	1.0	.5	1.3	1	[illegible]

[1] Ratio of moisture to aggregate by weight.

Group 3 shows the effect of the additives in improving the ability of specimens to retain compressive strength after having been subjected to a 74-day period of alternate immersion in water and air drying. Here, as in group 2, the additives were much more effective when used in the mixes containing granite No. 2. As in group 2, additive B was more effective than the other two additives. The effectiveness of additive B is emphasized further by the fact that, when used with granite No. 2, it produced a higher compressive strength after a 74-day period of wetting and drying than it did after the same period of straight air curing.

In the granite No. 2 mixes, additive D was more effective than additive A but, in the gravel No. 1 mixes, neither was highly effective since both additives failed to produce compressive strengths much greater than those obtained without additive.

CONCLUSIONS

In summarizing the results of this investigation the following statements appear to be warranted:

1. Additives are available that facilitate the coating of wet aggregate with liquid bituminous materials.
2. Additives are available that greatly increase the resistance of bituminous films to stripping in the presence of moisture.
3. The use of additives with tar does not appear to be warranted except possibly when mixing must be done under particularly adverse conditions, such as during a rain or immediately prior thereto.
4. Additives appear to have no detrimental effect upon the bituminous material with which they are used.
5. Additives appear to have no material effect on the original compressive strength of bituminous mixtures.
6. Additives are available that will reduce the loss of stability of bituminous mixtures due to the detrimental action of water.
7. The benefits afforded by additives appear to have considerable permanence but the degree of permanence was not disclosed by these tests.
8. The immersion-compression test provides a useful measure of the benefits to be derived from the use of additives.

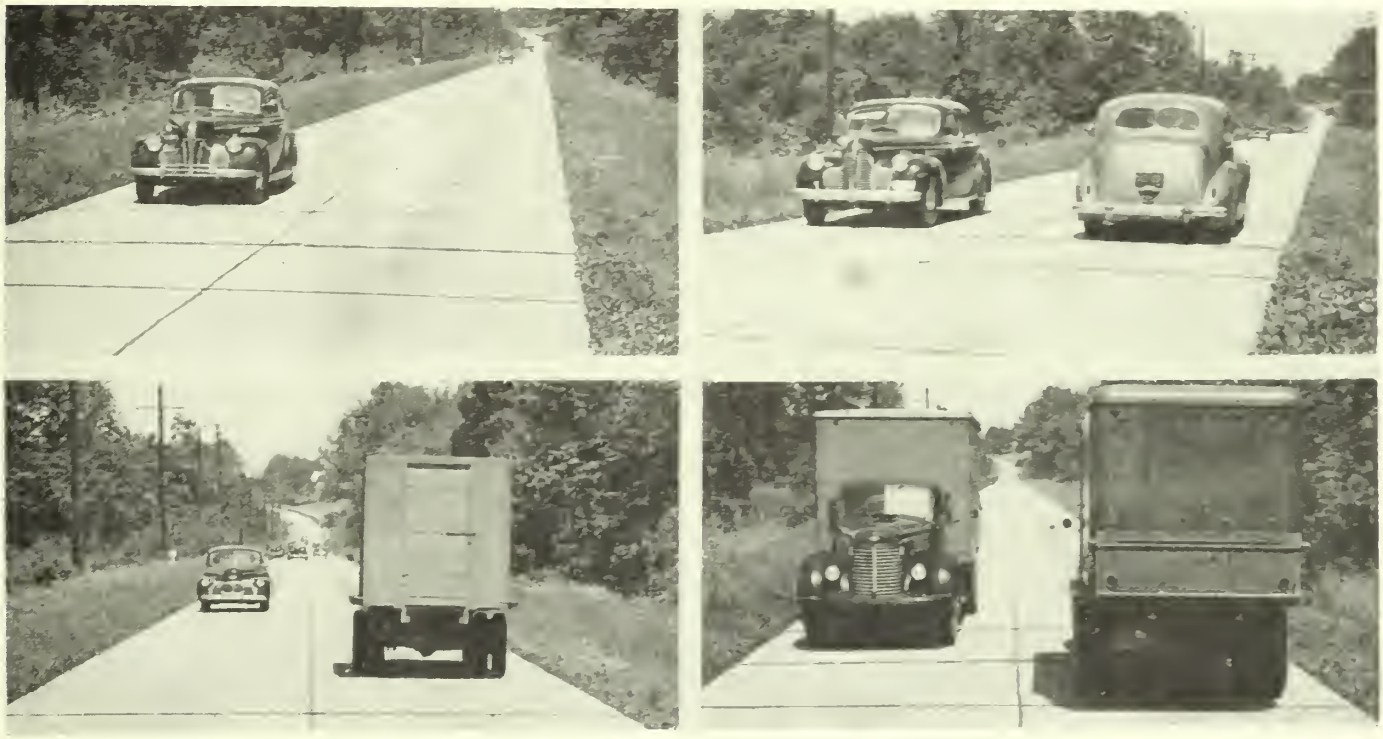
U S GOVERNMENT PRINTING OFFICE: 1945

EFFECT OF ROADWAY WIDTH ON VEHICLE OPERATION

A STUDY OF SPEED AND PLACEMENT ON TWO-LANE CONCRETE ROADS

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by A. TARAGIN, Highway Engineer



VEHICLE POSITIONS ON A TYPICAL 22-FOOT HIGHWAY.

TWO-LANE ROADS constitute 95 percent of the 256,000 miles of dustless surfaces on primary systems of rural State highways in this country. Nearly one-third of these two-lane roads have a portland cement concrete pavement. A survey of the surface widths of rural highways with traffic of 1,000 or more vehicles per day in 27 States shows that 48 percent of the roads at present have pavement widths less than 20 feet.

To determine operating conditions on surfaces of various widths, extensive speed-placement studies were conducted during immediate prewar years by the Public Roads Administration with the cooperation of the highway planning surveys in 10 States. Preliminary results of these studies on several sections of highway in Illinois were published in the 1943 Proceedings of the Highway Research Board. This report presents the complete analysis of all the data collected in the speed-placement studies on straight, level sections of rural two-lane concrete pavement.

The equipment used in obtaining the field data consisted of combination speed meters and transverse placement detectors, described in detail in the April 1940 issue of PUBLIC ROADS.[1] With this equipment the

TABLE 1.—*Study locations and vehicles included in analysis of speed-placement data*

CLASSIFIED BY STATE

State or pavement width	Day		Night		Total vehicles
	Locations	Vehicles	Locations	Vehicles	
	Number	*Number*	*Number*	*Number*	*Number*
California	2	11,502	.....	..	11,502
Illinois	9	23,890	4	4,847	28,737
Iowa	11	14,012	11	978	19,990
Maryland	2	1,216	2	605	1,821
Massachusetts	4	6,446	1	959	7,405
Minnesota	6	4,871	5	3,384	8,255
Ohio	3	2,811	2	238	3,049
Oregon	1	628	.....	.....	628
Texas	6	10,365	3	1,119	11,484
Washington	3	2,718	.....	.....	2,718
Total	47	78,439	28	17,150	95,589

CLASSIFIED BY PAVEMENT WIDTH

18 feet	15	24,961	6	4,511	29,472
20 feet	28	49,434	20	11,804	61,238
22 feet	3	1,844	2	605	2,449
24 feet	1	2,230	.....	.....	2,230
Total	47	78,439	28	17,150	95,589

[1] New Techniques in Traffic Behavior Studies, by E. H. Holmes and S. E. Reymer, PUBLIC ROADS, April 1940.

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transverse position and speed of each vehicle on the highway were recorded simultaneously on graphic recorder charts. Three charts, one for placements and one for speeds in each direction of travel, were synchronized and moved at a uniform speed so that it was possible to determine also the time spacing between successive vehicles, whether they were traveling in the same or in opposite directions.

VARIETY OF ROADWAY WIDTHS INCLUDED

The number of sections where studies were made and vehicles observed, classified by the State in which the study was made, and by lighting conditions, are shown in table 1. Speed and placement information for over 95,000 vehicles was obtained at 47 different locations on 18-, 20-, 22-, and 24-foot concrete pavements. At all 47 sites studies were conducted during the day, and at 28 of them observations were also made during several hours of darkness.

Surface widths of 18 and 20 feet are well represented in the sample, but the wider surfaces are less adequately represented because of the difficulty of finding suitable pavements of such widths in rural areas.

AMPLE SIGHT DISTANCES AT ALL LOCATIONS

All the sections were on State routes, and 38 of the sections were on U S numbered highways. Over 65 percent of the locations were at places where the sight distance exceeded 2,100 feet, the minimum passing sight distance for a design speed of 60 miles per hour, as recommended by the American Association of State Highway Officials.[2] All data were taken on sections where the sight distance was 1,200 feet or greater.

At all locations the pavement was flanked by well-maintained grass, gravel, or bituminous shoulders 4 to 10 feet in width. There were lip curbs and grass shoulders on six of the sections with 20-foot surfaces. The data for the lip-curb sections have not been combined with the data for sections without lip curbs.

A center line painted black, white, or yellow separated the two traffic lanes at all of the locations except one section of pavement 18 feet wide in Massachusetts, which had no lane markings. Placement results for this section are omitted from all the averages presented.

Table 2 shows highway conditions and average vehicle placements observed at the locations selected. Speeds and transverse positions are shown for passenger cars and commercial vehicles. Light delivery trucks and station wagons are included with passenger cars, while busses are grouped with commercial vehicles. Data are shown separately for day and night observations and by pavement widths.

The average placement figures include those for passing vehicles traveling in the left lane. Consequently they are influenced considerably by the relative number of passing vehicles and should not be compared to form conclusions as to transverse placements on pavements of various widths or with different types of shoulder.

VEHICLES GROUPED ACCORDING TO INFLUENCE OF OTHER VEHICLES

In order to permit true evaluation of the effect of various surface widths and types of shoulder on vehicle placement for different densities and compositions of traffic, data were segregated into the following groups representative of various positions of vehicles in the traffic stream:

[2] A Policy on Sight Distance for Highways, American Association of State Highway Officials, 1940, p. 25.

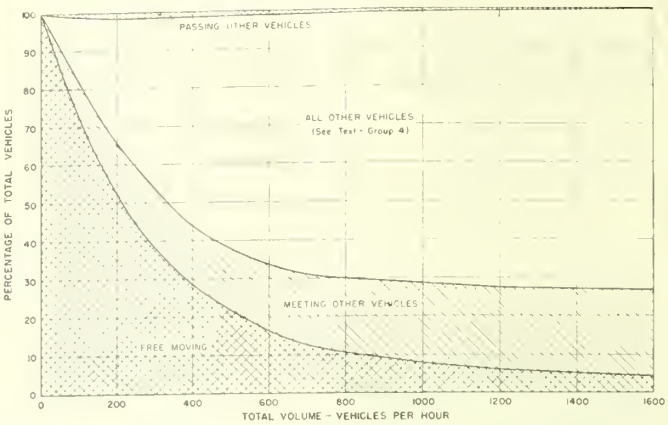


FIGURE 1.—DISTRIBUTION OF VEHICLES BY FREE-MOVING, MEETING, AND PASSING GROUPS ON TWO-LANE CONCRETE PAVEMENT WITH TRAFFIC EVENLY DISTRIBUTED IN EACH DIRECTION OF TRAVEL.

1. *Free-moving vehicles.*—Includes those vehicles that crossed the placement detector at least 6 seconds after any vehicle traveling in the same direction, and at least 5 seconds after and 10 seconds prior to passage across the detector of any vehicle traveling in the opposite direction.

Drivers in this group were, for practical purposes, uninfluenced by other traffic on the highway when speed and transverse position were recorded.

2. *Meeting vehicles.*—Includes those vehicles that were spaced more than 6 seconds behind any vehicle traveling in the same direction and either had met, or were to meet within 1.5 seconds, a vehicle traveling in the opposite direction.

The term "meeting vehicles," as defined, applies only to those vehicles that might have been directly affected by opposing traffic and were not in any way affected by vehicles traveling in the same direction. The transverse positions and the clearances between the bodies of vehicles as they meet are major factors in determining desirable roadway width for safe traffic operation.

3. *Passing vehicles.*—Includes those vehicles that were overtaking and passing other vehicles traveling in the same direction and which crossed the placement detector within 1 second of the passed vehicle. Wherever the term "passing" is used in this report, overtaking and passing is implied.

4. *All other vehicles.*—Includes those vehicles that were spaced less than 6 seconds from the preceding vehicle traveling in the same direction regardless of opposing traffic, and those vehicles that were 6 seconds or more from the preceding vehicle traveling in the same direction but which were so spaced with respect to opposing traffic that they could not be classified either as meeting or free moving.

To illustrate the significance of these special classifications, the distribution of vehicles by the four groups is shown in figure 1 for volumes up to 1,600 vehicles per hour when traffic was evenly distributed in each direction of travel. The percentage of free-moving vehicles dropped very rapidly as the volume increased. At a volume of 200 vehicles per hour the behavior of over half of the vehicles was uninfluenced by other vehicles at the point of observation. Only 10 percent of the vehicles were uninfluenced at a volume of 800 vehicles per hour.

The percentage of vehicles that were meeting other

TABLE 2.—*Speed, placement, and volume of all traffic on two-lane level tangent sections of concrete pavements*

18-FOOT WIDTH

State	Route number	Study section number	Shoulders		Average volume	Number of vehicles			Average speed			Distance of left wheels from center line		
			Width	Type		Passenger cars	Trucks	Total	Passenger cars	Trucks	Total	Passenger cars	Trucks	Total
Daytime:			*Feet*		*Vehicles per hr.*				*Miles per hr.*	*Miles per hr.*	*Miles per hr.*	*Feet*	*Feet*	*Feet*
Illinois	U S 12	1	5	Grass	555	4,646	70	4,716	41.4	38.9	41.4	1.4	1.0	1.4
Minnesota	S R 100	2	6	Grass	117	311	57	368	38.8	36.6	38.5	1.3	1.4	1.3
	U S 10	3	6	Grass	383	1,300	41	1,341	43.7	40.4	43.6	1.4	1.3	1.4
Iowa	U S 30	4	8	Grass	165	1,457	214	1,671	49.5	40.3	48.3	1.4	1.4	1.4
	U S 30	5	8	Grass	173	1,478	255	1,733	47.0	40.4	46.0	1.2	1.0	1.2
Illinois	S R 13	6	10	Grass	323	819	711	1,530	40.0	34.0	37.2	1.4	1.2	1.3
Illinois	U S 14	7	5	Gravel	268	1,286	88	1,374	46.5	37.1	45.9	1.6	1.7	1.6
	U S 14	8	6	Gravel	440	2,549	51	2,600	44.5	38.1	44.4	1.3	1.5	1.3
Washington	U S 101	9	5	Gravel	121	662	126	788	42.5	38.0	41.8	1.6	1.8	1.6
Massachusetts [1]	S R 114	10	5–6	Gravel	230	1,043	113	1,156	39.1	35.8	38.8	.6	.3	.6
Texas	U S 81	11	6	Gravel	108	407	118	525	44.8	42.3	44.2	1.1	1.5	1.2
Illinois	U S 45	12	5	Bituminous	685	4,336	36	4,372	39.5	35.9	39.5	1.6	1.8	1.6
Iowa	U S 61	13	6	Bituminous	155	433	83	516	48.2	42.5	47.3	1.5	1.6	1.5
Texas	U S 81	14	2+12	Bituminous and caliche.	214	1,189	224	1,413	45.7	40.1	44.8	1.7	2.2	1.8
	U S 83	15	7	Clay	130	717	131	848	42.5	34.8	41.3	1.8	2.0	1.8
Nighttime:														
Illinois	U S 12	1	5	Grass	604	2,174	32	2,206	39.5	41.5	39.5	1.1	.6	1.1
Minnesota	U S 10	3	6	Grass	357	855	20	875	41.6	37.5	41.5	1.5	1.2	1.5
Iowa	U S 30	4	8	Grass	79	570	95	665	45.0	37.6	43.9	1.4	.9	1.3
Iowa	U S 30	5	8	Grass	68	506	105	611	44.3	40.5	43.7	1.6	.6	1.4
Texas	U S 81	11	6	Gravel	73	61	12	73	45.6	41.1	44.9	1.4	1.5	1.4
Iowa	U S 61	13	6	Bituminous	85	256	25	281	45.0	36.9	44.3	1.4	1.3	1.4

20-FOOT WIDTH

Daytime:														
Iowa	S R 60	16	4	Grass	112	645	187	832	38.4	35.2	37.7	1.6	1.6	1.6
Illinois	U S 12	17	5	Grass	580	4,081	104	4,185	35.0	33.2	35.0	1.8	2.0	1.8
Iowa	U S 169	18	6	Grass	74	702	142	844	50.6	41.5	49.1	1.4	1.6	1.4
Ohio	U S 62	19	6	Grass	128	468	75	543	46.0	40.7	45.3	1.2	1.1	1.2
Texas	U S 83	20	6	Grass	283	2,376	469	2,845	40.3	35.9	39.6	1.7	2.1	1.8
Iowa	U S 69	21	8	Grass	199	1,673	261	1,934	48.5	41.2	47.5	1.9	1.9	1.9
	U S 64	22	8	Grass	121	1,141	183	1,324	49.4	41.4	48.3	1.2	1.3	1.2
Minnesota	U S 12	23	[2] 8	Grass	361	1,193	22	1,215	41.0	35.6	40.9	1.8	2.2	1.8
	U S 169	24	8	Grass	131	310	51	361	49.1	40.7	47.9	1.0	1.5	1.1
	U S 169	25	8	Grass	108	169	37	206	42.4	41.3	42.2	1.6	2.0	1.7
	U S 12	26	8	Grass	488	1,356	24	1,380	37.0	34.4	37.0	1.6	1.8	1.6
Washington	U S 410	27	8–12	Grass	182	841	231	1,072	42.7	39.4	42.0	1.6	1.7	1.6
Illinois	S R 13	28	10	Grass	299	336	37	373	39.3	30.8	38.5	1.8	2.0	1.8
Texas	U S 75	29	10	Grass	180	953	219	1,172	43.9	37.3	42.7	1.6	2.0	1.7
Illinois	U S 34	30	5	Gravel	257	2,215	295	2,510	34.7	29.0	34.0	1.9	1.9	1.9
Texas	U S 81	31	6	Gravel	345	3,307	255	3,562	44.0	37.8	43.6	2.0	2.2	2.0
Washington	U S 99	32	8	Gravel	275	721	137	858	45.3	40.9	44.6	1.7	1.7	1.7
Massachusetts	S R 28	33	2	Bituminous	371	1,482	169	1,651	37.0	35.1	36.8	2.4	2.3	2.4
	S R 28	34	3	Bituminous	272	935	128	1,063	37.8	34.8	37.4	2.0	1.8	2.0
California	U S 99	35	6	Bituminous	249	3,876	985	4,861	44.9	39.6	43.8	2.2	2.1	2.2
Massachusetts	U S 44	36	2+4	Bituminous and gravel.	372	2,356	220	2,576	40.3	37.8	40.1	1.9	1.9	1.9
California	U S 40	37	2+8	....do	629	6,288	353	6,641	40.0	38.6	39.9	2.4	2.2	2.4
Daytime, pavements with lip curbs:														
Iowa	U S 65	38	6	Grass	120	1,012	174	1,186	51.2	42.5	49.9	1.2	1.3	1.2
	U S 6	39	6	Grass	176	1,078	195	1,273	47.8	43.3	47.1	1.3	1.4	1.3
	S R 92	40	6	Grass	88	416	161	577	46.2	39.4	44.3	1.2	1.2	1.2
	U S 69	41	8	Grass	218	1,834	288	2,122	47.3	40.1	46.3	1.6	1.5	1.6
Ohio	U S 62	42	6	Grass	228	685	125	810	45.5	39.8	44.6	1.7	1.3	1.6
	S R 4	43	6	Grass	296	1,275	183	1,458	46.7	41.9	46.1	1.4	1.5	1.4
Nighttime:														
Iowa	S R 60	16	4	Grass	85	435	77	512	33.9	33.0	33.8	1.6	1.4	1.6
Illinois	U S 12	17	5	Grass	507	1,986	16	2,002	34.6	32.8	34.6	2.0	1.5	2.0
Iowa	U S 169	18	6	Grass	41	322	46	368	46.1	41.1	45.5	1.3	1.3	1.3
Ohio	U S 62	19	6	Grass	51	67	10	77	43.4	39.0	42.8	.8	1.1	.8
Texas	U S 83	20	6	Grass	201	414	35	449	41.8	36.3	41.4	1.8	2.2	1.8
Iowa	U S 69	21	8	Grass	85	556	78	634	42.2	40.1	41.9	1.8	1.1	1.7
	U S 64	22	8	Grass	56	402	97	499	45.2	40.6	44.3	.9	.7	.9
Minnesota	U S 12	23	[2] 8	Grass	469	1,283	21	1,304	42.1	37.5	42.0	1.7	2.4	1.7
	U S 169	24	8	Grass	96	272	33	305	49.2	39.8	48.2	1.1	1.5	1.1
	U S 169	25	8	Grass	51	153	42	195	41.1	37.3	40.3	1.3	1.2	1.3
	U S 12	26	8	Grass	237	694	11	705	41.8	34.9	41.7	1.5	1.5	1.6
Illinois	S R 13	28	10	Grass	265	409	23	432	40.9	36.0	40.6	1.8	1.6	1.8
Illinois	U S 34	30	5	Gravel	269	206	1	207	32.0	27.5	32.0	1.9	2.5	1.9
Texas	U S 81	31	6	Gravel	298	573	24	597	45.5	38.1	45.2	2.1	2.0	2.1
Massachusetts	U S 44	36	2+4	Bituminous and gravel.	588	942	17	959	34.8	36.3	34.8	1.7	2.6	1.7
Nighttime, pavement with lip curbs:														
Iowa	U S 65	38	6	Grass	65	206	40	246	48.1	42.2	47.1	1.0	.9	1.0
	U S 6	39	6	Grass	154	845	76	921	40.4	39.6	40.3	1.4	1.0	1.4
	S R 92	40	6	Grass	45	224	51	275	44.3	38.5	43.2	1.1	.5	1.0
	U S 69	41	8	Grass	161	851	115	966	43.4	39.4	42.9	1.6	1.1	1.5
Ohio	S R 4	43	6	Grass	161	140	21	161	41.9	34.9	41.0	1.4	.6	1.3

[1] No center-line markings.
[2] Eight feet of grass on one shoulder only. The other shoulder had 5 feet of bituminous mix and 3 feet of grass. Speed and placement were the same for both sides of road.

TABLE 2.—*Speed, placement, and volume of all traffic on two-lane level tangent sections of concrete pavements*—Continued

22-FOOT WIDTH

State	Route number	Study section number	Shoulders		Average volume	Number of vehicles			Average speed			Distance of left wheels from center line		
			Width	Type		Passenger cars	Trucks	Total	Passenger cars	Trucks	Total	Passenger cars	Trucks	Total
			Feet		*Vehicles per hr.*				*Miles per hr.*	*Miles per hr.*	*Miles per hr.*	*Feet*	*Feet*	*Feet*
Daytime:														
Maryland	S R 586	44	7	Gravel	170	387	38	425	45.8	40.5	45.3	2.2	2.1	2.2
Oregon	U S 99	45	8	Gravel	187	557	71	628	48.7	42.4	48.0	2.3	2.5	2.3
Maryland	U S 522	46	10	Clay	203	764	27	791	49.8	42.5	49.6	2.1	1.6	2.1
Nighttime:														
Maryland	S R 586	44	7	Gravel	96	290	1	291	48.0	45.0	48.0	1.8	1.5	1.8
	U S 522	46	10	Clay	126	273	41	314	47.5	39.0	46.4	2.0	1.6	1.9
24-FOOT WIDTH														
Daytime:														
Illinois	U S 67	47	10	Grass	334	1,805	425	2,230	41.5	35.2	40.3	2.5	2.5	2.5

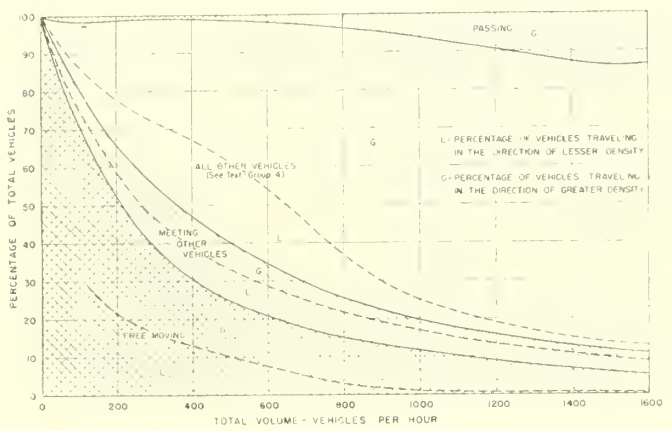

PASSING
G
L-PERCENTAGE OF VEHICLES TRAVELING IN THE DIRECTION OF LESSER DENSITY
G-PERCENTAGE OF VEHICLES TRAVELING IN THE DIRECTION OF GREATER DENSITY
ALL OTHER VEHICLES (See text Group 4)
MEETING OTHER VEHICLES
FREE MOVING
PERCENTAGE OF TOTAL VEHICLES
TOTAL VOLUME - VEHICLES PER HOUR

FIGURE 2.—DISTRIBUTION OF VEHICLES BY FREE-MOVING, MEETING, AND PASSING GROUPS ON TWO-LANE CONCRETE PAVEMENT WITH TRAFFIC PREDOMINANTLY HEAVY IN ONE DIRECTION (AVERAGE VOLUME IN DIRECTION OF LESS TRAVEL IS 120 VEHICLES PER HOUR).

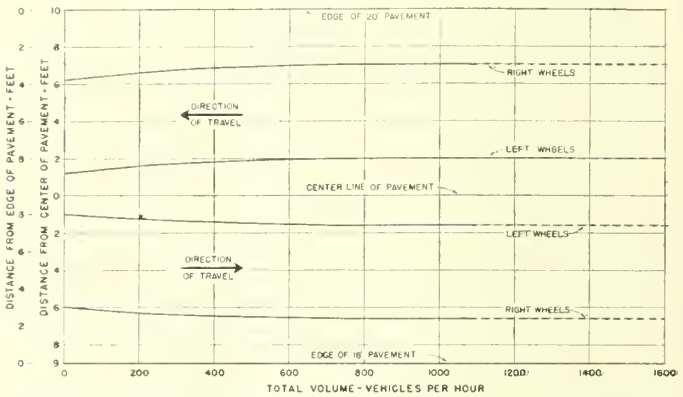


FIGURE 3.—EFFECT OF VOLUME ON TRANSVERSE POSITION OF ALL PASSENGER CARS ON 18- AND 20-FOOT CONCRETE PAVEMENTS WITH GRASS SHOULDERS.

vehicles, but were not within 6 seconds of the vehicle ahead, gradually increased with volume. Percentage of passings was highest at a volume of about 300 vehicles per hour, tapering off at the lower volumes because of the lessened necessity for passing, and at the higher volumes when the increased traffic in the opposite direction afforded fewer opportunities to pass. Although the percentage of passing vehicles may appear to be rather low, it should be remembered that in this analysis the two vehicles involved, the passing and the one being passed, had to be traveling within 1 second of each other at the observation point to be included in this category. Vehicles that were in earlier or later stages of passing maneuvers were included in the all-other-vehicles category.

Figure 2 shows a road where traffic was predominantly heavy in one direction of travel. The number of free-moving vehicles did not decrease as rapidly as on roads with the volume evenly distributed in the two directions. The percentage of meeting vehicles, as defined in this report,[2] was highest at a volume of about 400 vehicles per hour. Fourteen percent of the vehicles were within 1 second of the vehicles they were overtaking or passing in the direction of heavy density when the total traffic volume reached 1,500 vehicles per hour.

At the higher volumes the majority of the vehicles are included in group 4, and data on them could not be used to study the effect of different cross sections on vehicle behavior because so many factors influenced their position on the highway. There were, however, few sections with a traffic volume in excess of 400 vehicles per hour.

TRAFFIC VOLUME AFFECTS TRANSVERSE POSITION OF VEHICLES

The effect of volume on speeds of vehicles has been definitely determined from extensive research on highway capacity.[3] In this report analysis is directed primarily toward determination of the effect of traffic volume on the transverse placement of motor vehicles.

The effect of volume on transverse position of the average passenger car traveling on 18- and on 20-foot pavements is shown by figure 3. As the traffic on an 18-foot pavement (lower half of figure) increased to a volume of 700 vehicles per hour the average distance of the left wheels of passenger cars from the center of the pavement also increased until it became 1.8 feet. A further increase in density had no effect on the average position of the car. On the 20-foot pavement (upper half of figure) the rate of change in the position of the average vehicle with volume was the same as on the 18-foot pavement. At least two-thirds of the added pavement width was utilized to increase edge clearance.

Figure 4 shows the effect of traffic volume on the transverse positions of free-moving and meeting vehicles on 18-foot concrete pavement flanked by grass shoul-

[2] Results of Highway-Capacity Studies, by O. K. Normann, PUBLIC ROADS June 1942, also Highway Research Board Proceedings, 1941.

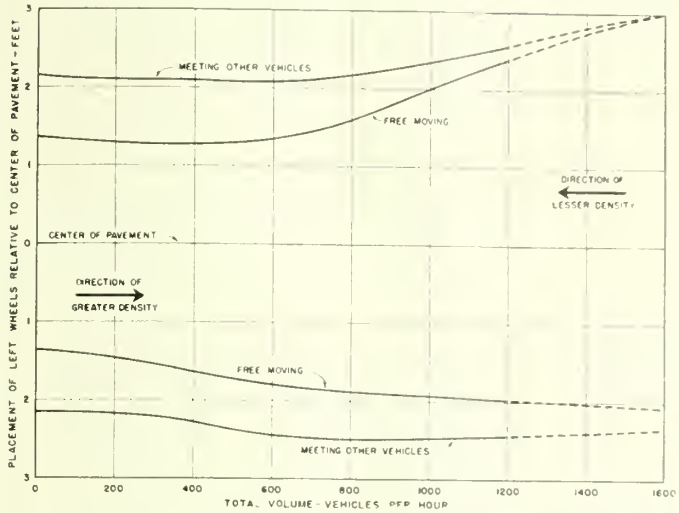

MEETING OTHER VEHICLES
FREE MOVING
DIRECTION OF LESSER DENSITY
CENTER OF PAVEMENT
DIRECTION OF GREATER DENSITY
FREE MOVING
MEETING OTHER VEHICLES
PLACEMENT OF LEFT WHEELS RELATIVE TO CENTER OF PAVEMENT - FEET
TOTAL VOLUME - VEHICLES PER HOUR

FIGURE 4.—EFFECT OF VOLUME ON TRANSVERSE PLACEMENT OF FREE-MOVING AND MEETING VEHICLES ON 18-FOOT CONCRETE PAVEMENT WITH GRASS SHOULDERS, AVERAGE DENSITY IN DIRECTION OF LESSER MOVEMENT, 120 MILES PER HOUR.

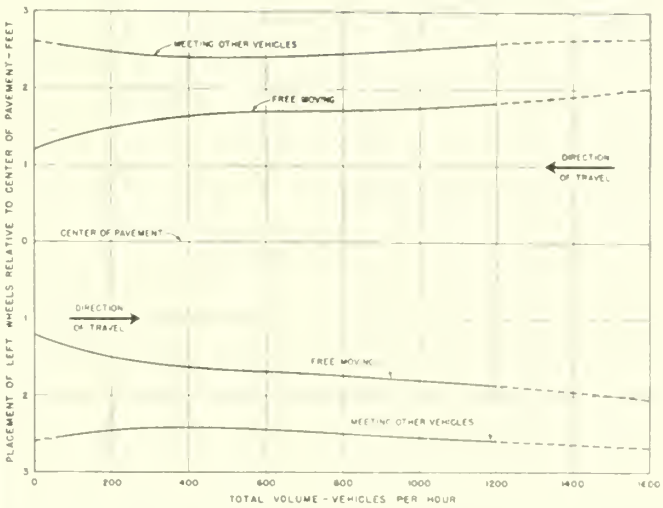

MEETING OTHER VEHICLES
FREE MOVING
DIRECTION OF TRAVEL
CENTER OF PAVEMENT
DIRECTION OF TRAVEL
FREE MOVING
MEETING OTHER VEHICLES
PLACEMENT OF LEFT WHEELS RELATIVE TO CENTER OF PAVEMENT - FEET
TOTAL VOLUME - VEHICLES PER HOUR

FIGURE 5.—EFFECT OF VOLUME ON TRANSVERSE PLACEMENT OF FREE-MOVING AND MEETING VEHICLES ON 20-FOOT CONCRETE PAVEMENT WITH GRASS SHOULDERS. TRAFFIC EVENLY DISTRIBUTED IN EACH DIRECTION OF TRAVEL.

ders. In this example traffic in the direction of lesser movement remained approximately uniform at 120 vehicles per hour, while that in the direction of greater movement increased. In the direction of heavier travel, the distance of the left wheels of the average free-moving vehicle from the center of the pavement increased from 1.5 to 2 feet, a change of five-tenths foot to the right, as the total volume increased from 200 to 1,200 vehicles per hour. Meeting vehicles altered their average position only three-tenths foot.

Free-moving vehicles in the direction of lesser movement shifted their position to the right, as the total volume increased, more than twice as much as the vehicles in the denser traffic stream, and during higher total volumes assumed almost the same position as when they met other vehicles. An increase in volume, therefore, causes a greater change in the position of free-moving vehicles than in the position of vehicles in any other group. This presumably was due to the fact that the length of time the free-moving vehicles remained free moving decreased with the increase in volume. Drivers shifted their vehicles from the free-moving position to the meeting position more frequently as the traffic density increased until the average driver found it more convenient to remain in a position almost coinciding with his position when meeting other vehicles. At a volume of 1,200 vehicles per hour the free-moving and meeting vehicles in the direction of lesser movement traveled with their left wheels 2.4 and 2.6 feet, respectively, from the center of the pavement.

The effect of volume on the transverse placement of free-moving and meeting vehicles on a 20-foot concrete pavement with traffic evenly distributed in each direction of travel is shown by figure 5. With an increase in volume from 200 to 1,200 vehicles per hour, free-moving vehicles moved to the right four-tenths foot, while meeting vehicles changed their transverse position only slightly as the volume increased.

SPEED UNAFFECTED BY PAVEMENT WIDTH

In an effort to determine the effect of pavement width on vehicle speed and the relation between the speed of a vehicle and its transverse position on the highway, average speeds were compiled for sections with the same surface width; and the average transverse placement for vehicles in each 5-mile-per-hour speed group was tabulated for each study site. Table 3 shows comparable speeds for two-lane roads having different pavement widths segregated according to vehicle classification and shoulder type for day and night operation.

TABLE 3.—*Average speeds on two-lane concrete roads*

DAYTIME

Vehicle type and traffic condition	18-foot pavement			20-foot pavement				22-foot pavement, gravel shoulder	24-foot pavement, grass shoulder
	Grass shoulder	Gravel shoulder	Bituminous shoulder	Grass shoulder		Gravel shoulder	Bituminous shoulder		
				Lip curb	No curb				
	Miles per hour	*Miles per hour*	*Miles per hour*	*Miles per hour*	*Miles per hour*	*Miles per hour*	*Miles per hour*	*Miles per hour*	*Miles per hour*
Passenger cars:									
Free moving	45.1	44.5	45.4	48.3	43.8	42.1	41.2	48.1	43.4
Meeting other vehicles	42.8	44.2	44.1	48.6	44.5	43.1	41.0	48.9	41.1
Passing passenger cars	52.1	51.5	54.4	59.3	54.0	51.3	46.5	52.5	46.3
Passing trucks or busses	52.5	48.5	50.0	51.4	57.6	50.0	52.1	67.5	45.5
Being passed	39.0	36.8	37.9	42.1	39.6	36.1	35.5	15.0	36.6
All, except passing	43.8	43.6	44.5	48.0	43.9	42.0	40.5	48.3	41.7
Trucks and busses:									
Free moving	38.6	38.3	38.4	41.1	37.9	35.6	37.6	40.6	36.0
Meeting other vehicles	38.8	39.5	38.9	41.8	38.4	37.5	37.4	45.9	34.4
Passing other vehicles	47.7	47.8	45.0	51.0	54.0	47.5	41.9	55.0	52.5
Being passed	33.5	30.8	33.1	37.7	35.5	31.2	32.2	37.5	42.5
All, except passing	38.5	38.6	38.2	41.1	37.0	35.9	37.2	41.8	35.1
Number of vehicles in sample	11,359	6,143	7,149	7,126	18,286	6,930	16,792	1,811	2,230

NIGHTTIME

Passenger cars:									
Free moving	43.9	45.5	45.7	41.2	43.4	40.1	37.1	47.9	
Meeting other vehicles	43.3	45.0	43.9	46.0	41.9	38.8	39.7	47.8	
Passing other vehicles	50.5	(1)	59.2	53.5	49.2	47.3	44.2	(1)	
Being passed	37.0	(1)	39.2	33.8	37.0	34.8	32.5	(1)	
All, except passing	42.8	46.3	45.4	43.7	42.1	39.2	35.0	47.7	
Trucks and busses:									
Free moving	40.0	43.0	36.4	39.3	37.6	38.7	37.5	42.5	
Meeting other vehicles	39.8	39.5	36.0	41.6	39.8	30.0	37.5	42.4	
Passing other vehicles	51.7	(1)	(1)	50.2	46.5	(1)	(1)	(1)	
Being passed	34.0	(1)	(1)	35.2	34.1	(1)	(1)	(1)	
All, except passing	38.9	41.1	36.9	38.9	36.6	32.4	36.3	42.0	
Number of vehicles in sample	4,357	73	281	2,569	7,182	801	959	605	

[1] Sample inadequate.

TABLE 4.—*Average speeds on pavements of various widths*

Type of vehicle and pavement width	Free-moving vehicles	Vehicles meeting oncoming traffic	All, except passing vehicles
	Miles per hour	*Miles per hour*	*Miles per hour*
Passenger cars during day:			
18 feet	44.9	43.4	43.7
20 feet	43.5	44.2	43.6
22 feet	48.1	48.9	48.3
24 feet	43.4	41.1	41.7
Total	44.4	44.3	44.0
Commercial vehicles during day:			
18 feet	38.5	39.1	38.5
20 feet	37.5	38.2	36.8
22 feet	40.6	45.9	41.8
24 feet	36.0	34.4	35.1
Total	38.1	39.1	37.8
Passenger cars at night:			
18 feet	45.2	43.6	43.5
20 feet	44.9	41.5	41.9
22 feet	47.9	47.8	47.7
Total	45.3	42.6	42.8
Commercial vehicles at night:			
18 feet	40.6	39.5	39.3
20 feet	37.7	38.4	36.0
22 feet	42.5	42.4	42.0
Total	38.8	39.0	37.4

Table 4 shows average speeds for the more important vehicle classifications on the different width surfaces for which the shoulders and other design features were comparable. For the sections included in this study on which vehicle speeds were typical of modern two-lane highways, pavement width apparently had no consistent effect on the average speeds of either the free-moving vehicles or those meeting oncoming traffic. It is interesting to note that the speed of the average driver is practically the same when meeting an oncoming vehicle as when uninfluenced by other vehicles. Evidently, drivers do not consider it necessary to reduce speed when approaching and meeting opposing traffic. Speeds at night are practically the same as daytime speeds.

TABLE 5.—*Average distance between left wheels and center line of typical 20-foot concrete pavements with different widths of grass shoulders*

State	Shoulder width	Passenger cars			Commercial vehicles		
		Free-moving	Meeting passenger cars	All, except passing vehicles	Free-moving	Meeting passenger	All, except passing vehicles
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Iowa	4	1.5	2.7	1.7	1.4	2.4	1.6
	6	1.4	2.6	1.5	1.6	2.2	1.6
	8	1.5	2.5	1.7	1.5	2.3	1.6
Illinois	5	1.6	2.4	1.9	1.8	2.4	1.8
	10	1.5	2.3	1.9	1.6	2.4	2.0
Texas	6	1.8	2.2	1.9	2.0	2.5	2.1
	10	1.6	2.5	1.8	1.8	2.6	2.0

Perhaps the most important consideration is that drivers did not travel more slowly on the narrower than on the wider surfaces. This should not be interpreted as implication that the narrower surfaces are as adequate for the desired speeds as are the wider roads. It is more likely that drivers maintain their desired speeds in the face of an apparently greater hazard on the narrow widths.

In analyzing speed-placement data for the several vehicle classification groups, no definite relation could be found between the speed of the vehicle and its trans-

TABLE 6.—*Average transverse placement of vehicles and clearance between meeting vehicles on two-lane concrete roads*

Vehicle classification	18-foot pavement			20-foot pavement				22-foot pavement	24-foot pavement
	Grass shoulder	Gravel shoulder	Bituminous shoulder	Grass shoulder		Gravel shoulder	Bituminous shoulder	Gravel shoulder	Grass shoulder
				Lip curb	No curb				
DISTANCE OF LEFT WHEEL FROM CENTER LINE DURING THE DAY									
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Passenger cars:									
Free moving	1.4	1.5	1.7	1.3	1.5	1.9	2.3	2.1	2.4
Meeting other passenger cars	2.1	2.1	2.5	2.3	2.5	2.6	3.0	2.9	3.1
Meeting trucks or busses	2.3	2.4	2.4	2.6	2.6	2.7	3.3	3.2	3.4
All, except passing	1.6	1.6	1.9	1.6	1.8	2.1	2.5	2.3	2.7
Trucks or busses:									
Free-moving	1.2	1.5	1.6	1.2	1.6	1.8	1.9	2.0	2.2
Meeting passenger cars	1.8	1.8	2.3	1.9	2.3	2.2	2.5	2.8	2.9
Meeting trucks or busses	1.5	2.2	2.5	2.7	2.5	2.5	2.5	2.9	3.0
All, except passing	1.2	1.6	1.9	1.4	1.8	1.9	2.1	2.1	2.5
DISTANCE OF LEFT WHEEL FROM CENTER LINE AT NIGHT									
Passenger cars:									
Free-moving	1.5	1.3	1.5	1.4	1.4	2.0	1.7	1.7	
Meeting other passenger cars	2.1	2.0	2.3	2.6	2.4	2.6	2.9	2.8	
Meeting trucks or busses	2.5	2.5	2.5	2.7	2.5	3.2	3.0	3.0	
All, except passing	1.6	1.4	1.7	1.5	1.6	2.1	1.8	2.0	
Trucks and busses:									
Free-moving	.8	1.4	1.1	.7	1.4	2.0	2.5	1.4	
Meeting passenger cars	1.3	1.5	2.0	1.7	1.9	2.2	3.0	2.4	
Meeting trucks or busses	1.7	(1)	(1)	1.8	2.0	2.5	(1)	(1)	
All, except passing	1.0	1.5	1.3	.8	1.5	2.2	2.6	1.6	
CLEARANCE BETWEEN BODIES OF MEETING VEHICLES DURING DAY									
Passenger cars meeting passenger cars	3.2	3.2	4.0	3.6	4.0	4.2	5.0	4.8	5.2
Passenger cars and trucks or busses meeting	2.6	2.7	3.2	3.0	3.4	3.4	4.3	4.5	4.8
Trucks or busses meeting trucks or busses	1.1	2.5	3.0	3.4	3.0	3.0	3.0	3.8	4.0
CLEARANCE BETWEEN BODIES OF MEETING VEHICLES AT NIGHT									
Passenger cars meeting passenger cars	3.3	3.0	3.6	4.3	3.9	4.3	4.8	4.5	
Passenger cars and trucks or busses meeting	2.3	2.5	3.0	2.9	2.9	4.0	4.5	3.9	
Trucks or busses meeting trucks or busses	1.5	(1)	(1)	1.6	2.1	3.0	(1)	(1)	

[1] Sample inadequate.

TABLE 7.—*Transverse placements of wheels and clearances between bodies of vehicles involved in passing maneuvers on two-lane concrete roads* [1]

PASSENGER CARS PASSING PASSENGER CARS [2]

Vehicle classification	Daytime									Nighttime						
	18-foot pavement			20-foot pavement				22-foot pavement	24-foot pavement	18-foot pavement			20-foot pavement			
	Grass shoulder	Gravel shoulder	Bituminous Shoulder	Grass shoulder		Gravel shoulder	Bituminous shoulder	Gravel shoulder	Grass shoulder	Grass shoulder	Gravel shoulder	Bituminous shoulder	Grass shoulder		Gravel shoulder	Bituminous shoulder
				Lip curb	No curb								Lip curb	No. curb		
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Passing passenger car	1.3	1.4	1.3	1.4	1.7	1.7	1.2	2.4	1.7	1.8	2.5	2.1	1.9	1.2	1.0	2.4
Passed passenger car	2.0	2.0	2.0	1.9	2.2	2.2	2.9	2.5	4.1	1.8	1.5	2.2	1.9	2.2	2.7	2.4
Body clearance	2.3	2.4	2.3	2.3	2.9	2.9	3.1	3.9	4.8	2.6	3.0	3.3	2.8	2.4	2.7	3.8
PASSENGER CARS PASSING TRUCKS OR BUSSES [2]																
Passing passenger car	2.4	2.2	2.5	2.1	1.8	2.3	2.3	1.8	1.7	1.0	------	------	0.8	2.0	------	------
Passed trucks or busses	1.3	1.8	2.0	1.5	1.9	1.9	2.0	2.5	2.8	1.5	------	------	1.5	1.8	------	------
Body clearance	2.2	2.5	3.0	2.1	2.2	2.7	2.8	2.8	3.0	1.0	[3]	[3]	.8	2.3	[3]	[3]
TRUCKS OR BUSSES PASSING PASSENGER CARS [2]																
Passing trucks or busses	1.2	1.5	.8	1.9	1.3	1.0	1.3	------	------	------	------	------	------	------	------	------
Passed passenger car	1.4	1.5	2.2	1.4	1.9	2.5	3.1	------	------	------	------	------	------	------	------	------
Body clearance	1.1	1.5	1.5	1.8	1.7	2.0	2.9	[3]	[3]	------	------	------	------	------	------	------
TRUCKS OR BUSSES PASSING TRUCKS OR BUSSES [2]																
Passing trucks or busses	3.0	1.5	1.3	2.5	1.2	------	1.3	------	------	------	------	------	------	------	------	------
Passed trucks or busses	1.0	1.5	1.9	1.5	2.5	------	2.0	------	------	------	------	------	------	------	------	------
Body clearance	2.0	1.0	1.2	2.0	1.7	[3]	1.3	[3]	[3]	------	------	------	------	------	------	------

[1] Passing vehicle within 1 second of passed vehicle.
[2] Distance that left wheel of passed vehicle was to the right of center line or distance that right wheel of passing vehicle was to the left of center line.
[3] Sample inadequate.

verse position on the highway at most of the observation points. At a few of the locations those drivers of free-moving passenger cars that maintained the higher speeds traveled somewhat closer to the center line of the highway than did drivers of slower moving vehicles. However, the difference between the transverse positions of vehicles in the fastest speed group and the slowest speed group was of small magnitude, generally below five-tenths foot. There was no difference in average placements between the speed groups for commercial vehicles or for passenger cars in any of the other classifications. It is apparent that the determination of an adequate pavement width for existing traffic speeds can be based on an analysis of the transverse positions on the various widths without regard to the relation between the speeds of individual vehicles and their transverse positions.

SHOULDER WIDTH IN EXCESS OF FOUR FEET DOES NOT AFFECT TRANSVERSE POSITION OF VEHICLES

While the field studies were in progress, a number of adjacent sections of road carrying the same traffic and having identical design features except for the shoulder width and type were selected for study to determine the effect of these variables on the average transverse position of vehicles.

Table 5 shows that comparable placement values for sections with 4- to 10-foot grass shoulders are practically identical. Shoulder width in excess of 4 feet does not seem to affect the transverse position of vehicles. This is also borne out by the fact that at some of the sections where the shoulders on one side of the pavements were wider than on the other side, the average position assumed was the same for both directions of travel. In no case were there any vertical obstructions immediately adjacent to the outside of the shoulder.

The fact that the portion of a shoulder more than 4 feet from the pavement edge does not affect the transverse position of vehicles has no relation to the need for providing shoulder of a width adequate to accommodate standing or moving vehicles entirely off the pavement in emergency.

Tables 6 and 7 show the average transverse placements of vehicles and clearances between the bodies of meeting and passing vehicles segregated by the various vehicle classifications and shoulder types for day and night operation. Values are averaged for all shoulder widths.

A majority of the shoulders on the highway sections included in this study were grass. A sufficient number of sections with gravel shoulders and bituminous shoulders were included to permit an analysis of the relative effect of such shoulders on the transverse positions of vehicles on concrete pavements 18 and 20 feet wide.

Table 8 shows, for each vehicle classification and for two surface widths, the difference between the average driver's position on a highway with grass shoulders and his position on a highway with gravel shoulders. A minus value is shown when the average driver traveled closer to the grass than to the gravel shoulder.

The table shows that the difference in placements for grass and gravel shoulders did not exceed one-tenth foot for 21 out of a total of 34 values representing the various vehicle classifications and the two surface widths. Most of the differences greater than one-tenth foot are for commercial vehicles or vehicles involved

TABLE 8.—*Difference between the transverse positions of vehicles on sections with grass and with gravel shoulders during day*

DIFFERENCE BETWEEN DISTANCES THAT VEHICLES WERE FROM RIGHT-HAND SHOULDERS [1]

Vehicle classification	18-foot surface	20-foot surface
	Feet	*Feet*
Passenger cars:		
Free moving	0.1	0.4
Meeting other passenger cars	0	.1
Meeting trucks or busses	.1	.1
Passing other passenger cars	.1	0
Passing trucks or busses	.2	−.5
Being passed by other passenger cars	0	0
All, except those passing other vehicles	0	.3
Trucks and busses:		
Free moving	.3	.2
Meeting passenger cars	0	−.1
Meeting other trucks or busses	(2)	0
Being passed by passenger cars	.5	0
All, except those passing other vehicles	.4	.1

DIFFERENCE BETWEEN CLEARANCES OF VEHICLE BODIES [1]

Passenger cars meeting other passenger cars	0	0.2
Passenger cars and trucks or busses meeting	.1	0
Passenger cars passing other passenger cars	.1	0
Passenger cars passing trucks or busses	.3	.5
Trucks or busses meeting other trucks or busses	(2)	0

[1] A minus sign is shown when the vehicle traveled closer to the grass than the gravel shoulder.
[2] Sample inadequate.

TABLE 9.—*Effect of bituminous shoulders and effect of increase in surface width on the transverse position of vehicles*

INCREASE IN THE DISTANCE BETWEEN VEHICLE AND CENTER LINE OF HIGHWAY

Vehicle classification	Bituminous shoulders compared with grass and gravel shoulders		Effect of increase in surface width on sections with grass or gravel shoulders	
	18-foot surface width	20-foot surface width	20-foot surface compared with 18-foot surface	22-foot surface compared with 20-foot surface
	Feet	*Feet*	*Feet*	*Feet*
Passenger cars:				
Free moving	0.3	0.7	0.2	0.2
Meeting other passenger cars	.4	.5	.4	.3
Meeting trucks or busses	.1	.7	.3	.5
Passing other passenger cars [1]	0	−.5	.4	.7
Passing trucks or busses [1]	.2	.4	−.4	.5
Being passed by other passenger cars	0	.6	.2	.3
All, except those passing other vehicles	.3	.7	.2	.2
Trucks or busses:				
Free moving	.3	.3	.3	.2
Meeting passenger cars	.5	.2	.5	.6
Meeting other trucks or busses	.7	0	.7	.4
Being passed by passenger cars	.5	.1	.4	.6
All, except those passing other vehicles	.5	.3	.4	.2

INCREASE IN CLEARANCE BETWEEN VEHICLE BODIES

Passenger cars meeting other passenger cars	0.8	1.0	0.8	0.6
Passenger cars and trucks or busses meeting	.6	.9	.8	1.1
Passenger cars passing other passenger cars	0	.1	.6	1.0
Passenger cars passing trucks or busses	.7	.5	0	.1
Trucks or busses meeting other trucks or busses	1.4	0	1.4	.8

[1] Increase in distance to left for passing vehicles, to right for all others.

in passing maneuvers which were represented by smaller samples than the other classifications.

Similar results were obtained by an analysis of placements at night on sections with grass and gravel shoulders. It may be concluded, that well-maintained grass shoulders have the same effect as well-maintained gravel shoulders on the transverse position of vehicles on two-lane concrete surfaces.

On highways with bituminous shoulders, the average driver traveled closer to the right-hand shoulder than on highways with grass or gravel shoulders. To evaluate this increase in the effective pavement width, table 9 has been prepared.

TABLE 10.—*Effect of lip curbs on the transverse positions of vehicles on 20-foot pavements with grass shoulders*

DIFFERENCE IN DISTANCE BETWEEN VEHICLE BODIES AND THE HIGHWAY CENTER LINE

Vehicle classification	20-foot pavement without curbs compared with 20-foot pavement with lip curbs	20-foot pavement compared with 18-foot pavement (both without curbs)
	Feet	*Feet*
Passenger cars:		
Free moving	0.2	0.1
Meeting other passenger cars	.2	.4
Meeting trucks or busses	0	.3
Passing other passenger cars	.3	.4
Passing trucks or busses	.3	−.6
Being passed by other passenger cars	.3	.2
All, except those passing other vehicles	.2	.2
Trucks and busses:		
Free moving	.4	.4
Meeting passenger cars	.4	.5
Meeting other trucks or busses	−.2	1.0
Being passed by passenger cars	.4	.2
All, except those passing other vehicles	.4	.6

DIFFERENCE IN CLEARANCE BETWEEN VEHICLE BODIES

Passenger cars meeting other passenger cars	0.4	0.8
Passenger cars and trucks or busses meeting	.4	.8
Passenger cars passing other passenger cars	.6	.6
Passenger cars passing trucks or busses	.1	0
Trucks or busses meeting other trucks or busses	−.4	1.9

Comparable values are shown for the effect that bituminous shoulders have on the transverse positions of vehicles and the effect that 2-foot wider surfaces have on the transverse positions. For example, the average driver of a free-moving passenger car traveled three-tenths foot farther to the right of the center line on 18-foot surfaces when there were bituminous shoulders than when the shoulders were grass or gravel. On highways with grass or gravel shoulders, he traveled two-tenths foot farther to the right of the center line when the surface width was 20 feet than he did when the surface width was 18 feet. The bituminous shoulder on the 18-foot surface had a greater effect on the transverse position of the free-moving passenger cars than an increase in surface width of 2 feet. Likewise, bituminous shoulders instead of grass or gravel shoulders improved the transverse position of free-moving passenger cars on 20-foot surfaces to a greater extent than did increasing surface width from 20 to 22 feet with grass or gravel shoulders. Also, bituminous shoulders induced as much increase in clearance between passenger cars meeting other passenger cars as did an increase of 2 feet in surface width.

Results of similar comparisons between the effect of bituminous shoulders and the effect of wider surfaces on the transverse position of vehicles under other traffic conditions are not always consistent, especially for those conditions represented by small samples. In general, however, bituminous shoulders at least 4 feet wide adjacent to 18- and 20-foot surfaces increased the effective surface widths during the day approximately 2 feet.

None of the pavements wider than 20 feet had bituminous shoulders, and comparatively little data were recorded at night on the 18- and 20-foot surfaces. An analysis of the daytime data and the limited nighttime data for 18- and 20-foot surfaces does indicate, however,

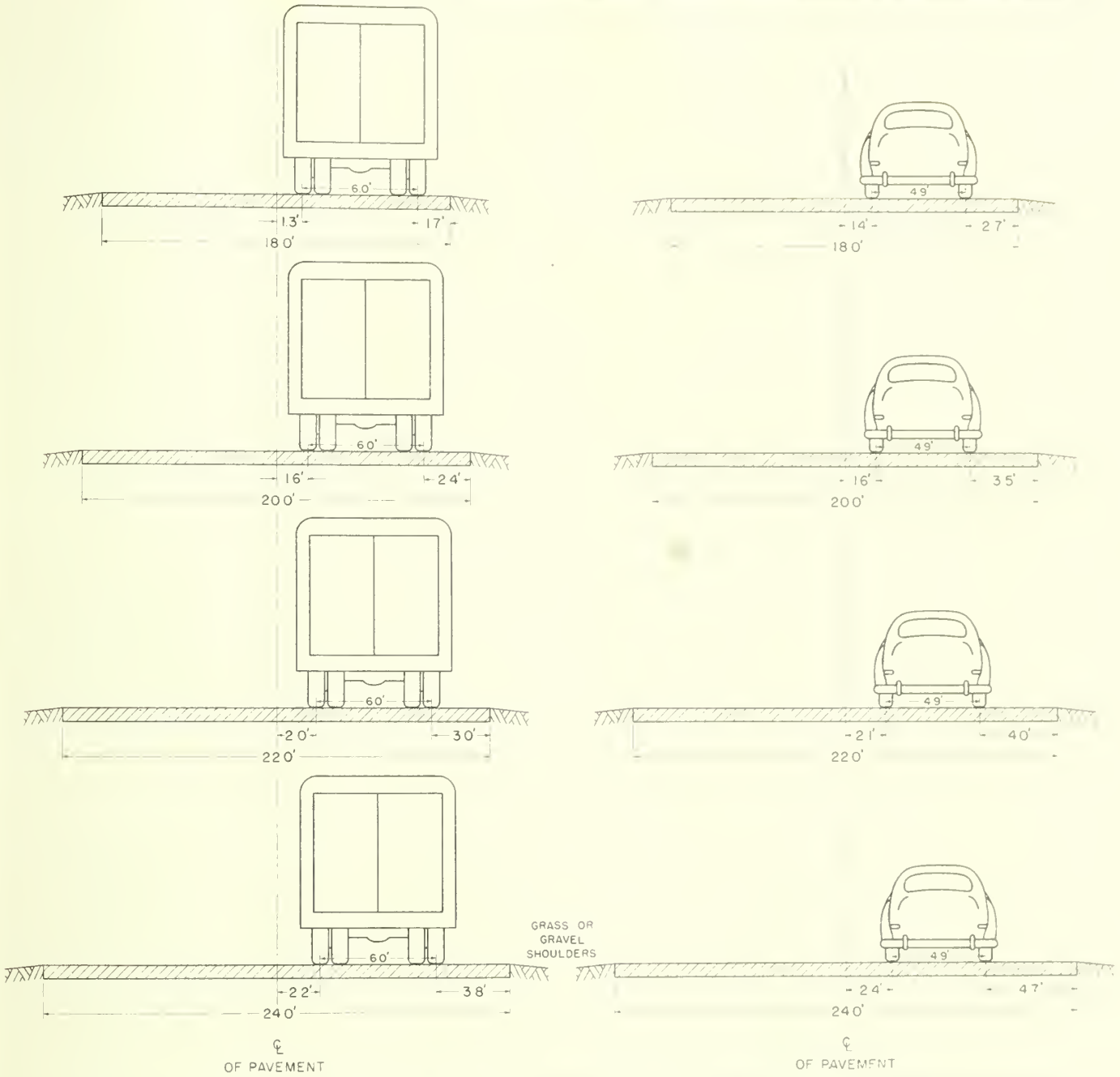


FIGURE 6.—AVERAGE POSITION OF FREE-MOVING VEHICLES ON TWO-LANE CONCRETE PAVEMENTS DURING THE DAY.

that bituminous shoulders increase the effective pavement width as much at night as during the day; and their effect on transverse positions of moving vehicles for the wider surfaces is somewhat less than for the narrower surfaces.

LIP CURBS REDUCE EFFECTIVE WIDTH

To determine the effect of lip curbs on vehicle position, a total of six locations on 20-foot concrete surfaces with lip curbs and comparable sections without curbs were selected for study in Iowa and Ohio. Driver-behavior data were recorded for a total of 10,000 vehicles on the lip-curb pavements and for a somewhat larger number of vehicles on the pavements without curbs.

Table 10 shows the effect of lip curbs on the transverse positions of vehicles on 20-foot pavements with grass shoulders for day operation. Vehicles under nearly all conditions traveled closer to the center of highways with lip curbs than on sections without lip curbs. The average reduction in distance from the center due to lip curbs was two-tenths foot for passenger cars and four-tenths foot for commercial vehicles. These differences are similar but somewhat less than those found when placements on 20-foot pavements without curbs were compared with placements on 18-foot pavements without curbs.

It may be noted that under some traffic conditions, the lip curbs had an effect the same as or greater than that of a reduction of 2 feet in pavement width. On an average, however, especially for the more important conditions that govern pavement width, including meeting vehicles, the effect of lip curbs is about half as great as that of a reduction of 2 feet in

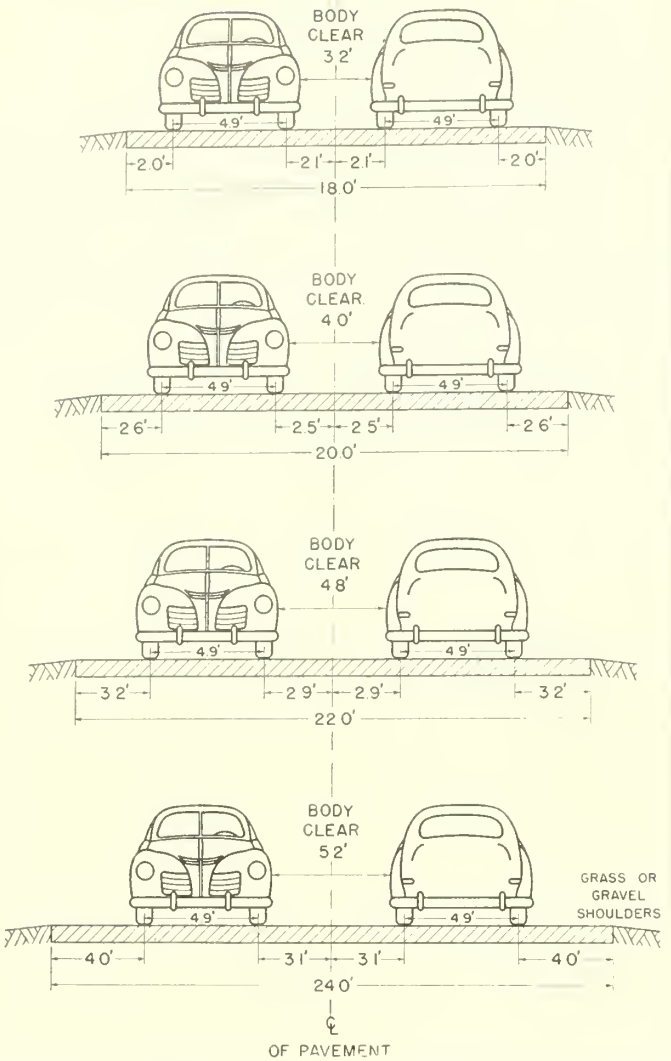


FIGURE 7.—AVERAGE POSITION OF PASSENGER CARS MEETING PASSENGER CARS ON TWO-LANE CONCRETE PAVEMENTS DURING THE DAY.

surface width. During the day, lip curbs reduce the effective pavement width approximately 1 foot.

Comparative data show that curbs affect the transverse position of commercial vehicles at night as much as during the day. Lip curbs apparently do not have any appreciable effect on the transverse position of passenger cars at night.

POSITION OF VEHICLES ON SURFACES OF VARIOUS WIDTH DETERMINED

Free-moving vehicles.—Figure 6 shows the average position of free-moving vehicles on two-lane concrete pavements during the day. In figure 6 and all the succeeding figures and tables the values shown are the average vehicle placements for highway sections with grass or gravel shoulders. This procedure is possible because the previous analysis showed no measurable difference between operating on pavements with grass shoulders and those with gravel shoulders.

Figure 6 shows values for passenger cars on the right and on the left are the values for commercial vehicles. The average width of bodies of passenger cars as obtained from measurements of various makes and models of cars was found to be 6 feet. A width of 8 feet for bodies of commercial vehicles is used. The tread widths (distance between centers of wheels) of 4.9 feet for passenger cars and 6 feet for trucks and busses were determined from the wheel placements recorded in this study.

Free-moving passenger cars and commercial vehicles steadily increase their distances from the center of the pavement as width of pavement increases. The added width provided by a 20-foot over an 18-foot pavement is utilized by drivers of free-moving passenger cars almost entirely to increase their distance from the pavement edge, while the distance from the center of the pavement increases only two-tenths foot or to a value of 1.6 feet. The added widths provided by 22- and 24-foot surfaces, however, are utilized to increase the distance from the center as well as from the edge. Drivers of commercial vehicles utilize the added widths more to increase the distance from the edge than to increase their distance from the center as pavements wider than 18 feet are traveled. For each foot of added lane width, drivers of commercial vehicles, on the average, increase the distance from the edge by seven-tenths foot and from the center by three-tenths foot.

Apparently, drivers judge the position of their vehicles when uninfluenced by other traffic, by distance from the center line, since this distance is about the same for both passenger cars and commercial vehicles on pavements of the same width. The narrower tread results in greater edge clearance for passenger cars than for trucks. Drivers of passenger cars keep the centers of their vehicles approximately 1 foot to the left of the center of their lane, while drivers of commercial vehicles keep the centers of their vehicles about five-tenths foot to the left. The distance between the center of the vehicle and the center of the lane increases, however, as wider pavements are traveled.

Passenger cars meeting passenger cars.—Figure 7 shows the average position of passenger cars meeting other passenger cars during the day. These vehicles travel approximately in the center of their respective lanes on the 18- and 20-foot pavements. On 18-foot surfaces drivers allow a wheel-track distance of 2 feet from the edge and 2.1 feet from the center of the road, thus allowing a body clearance between vehicles of 3.2 feet. When 20- and 22-foot pavements are traveled, the added width is used to increase both the clearance between vehicles and the distance from the edge. However, the additional pavement width provided by a 24-foot surface is used almost entirely to increase the distance from the edge. This indicates that the desired clearance between bodies of meeting passenger cars is about 5 feet, since this clearance is nearly obtained on 22-foot surfaces and does not increase materially on 24-foot surfaces.

Passenger cars meeting commercial vehicles.—Figure 8 shows the average position of passenger cars and commercial vehicles when meeting. The fact that passenger cars on 18- and 20-foot pavements do not move to the right to allow greater clearance when meeting commercial vehicles than when meeting passenger cars is a strong indication that even 20-foot roads are entirely too narrow when there is considerable truck traffic. Average body clearances of 2.6 and 3.5 feet for passenger cars meeting commercial vehicles on 18- and 20-foot pavements, respectively, appear to be inadequate for safety. An analysis of the distribution of vehicle clearances showed that clearance was 1 foot or less for 12 percent of meetings on 18-foot pavement and for 5 per-

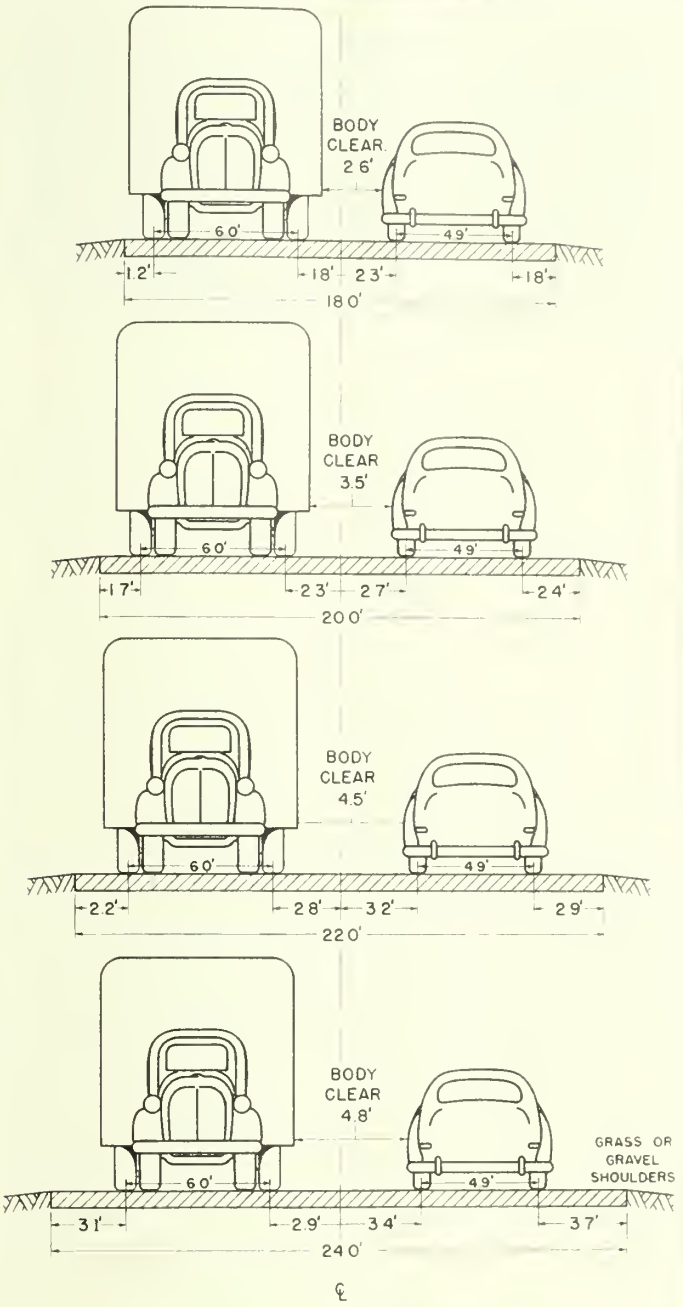


FIGURE 8.—AVERAGE POSITION OF PASSENGER CARS AND COMMERCIAL VEHICLES MEETING ON TWO-LANE CONCRETE PAVEMENTS DURING THE DAY.

cent of those on 20-foot pavements. Clearances of 2 feet or less were allowed by 47 percent of the drivers on 18-foot pavements and by 11 percent on 20-foot pavements.

The influence exerted by passenger cars on the transverse position of vehicles traveling in the opposite direction is shown by figure 9. On the 18-foot pavement, when passenger cars meeting other vehicles travel 1 foot to the left of the center line, and a small but significant number of cars do assume this position, the average position of the vehicles being met is 2.7 feet to the right of the center line. This allows a clearance of only seven-tenths foot between the bodies of the meeting vehicles. Similarly, on a 20-foot pavement, when passenger cars that encroach on the left lane meet other vehicles, the body clearances are only about 1 foot.

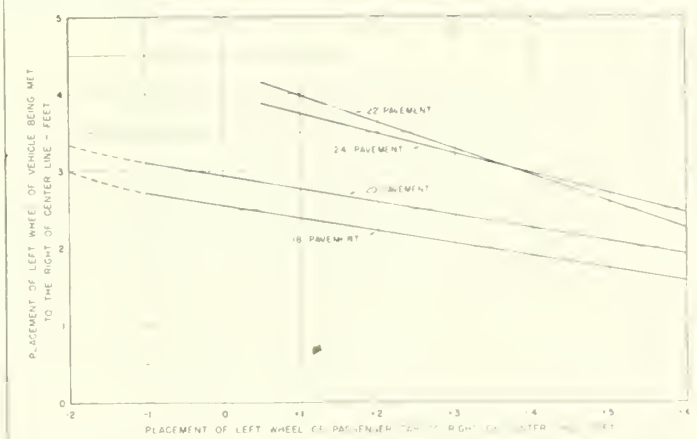

PLACEMENT OF LEFT WHEEL OF VEHICLE BEING MET TO THE RIGHT OF CENTER LINE - FEET
22' PAVEMENT
24' PAVEMENT
18' PAVEMENT
PLACEMENT OF LEFT WHEEL OF PASSENGER CAR TO RIGHT OF CENTER LINE - FEET

FIGURE 9.—EFFECT OF TRANSVERSE POSITION OF PASSENGER CARS ON THE POSITION OF THE MEETING VEHICLES—CONCRETE PAVEMENT WITH GRASS OR GRAVEL SHOULDERS.

Such clearances are entirely inadequate for safe operation.

On 22- and 24-foot pavements, passenger-car clearances are considerably greater than on 20-foot pavement. When passenger cars are five-tenths foot to the right of the center line on 22-foot pavements, vehicles they meet travel 4 feet on the other side of the center line, thus allowing a body clearance of 3.5 feet.

Commercial vehicles meeting commercial vehicles.—Figure 10 shows the average position of commercial vehicles when meeting other commercial vehicles during the day. Eighteen-foot pavements are so inadequate for these wide vehicles that the average body clearance is only 1.6 feet.

Although truck drivers travel closer to the edge of the pavement than do passenger-car drivers, they also sacrifice center clearance to avoid too little edge clearance. On 18-foot pavements the average truck driver, on meeting another truck, does not travel closer to the edge than 1.2 feet. The additional width provided by the 20-foot surface is used mostly for additional clearance between vehicles, whereas the increase from 22 to 24 feet is utilized to increase the distance from the pavement edge. Only on the 24-foot surface do commercial vehicles travel in the center of their respective lanes.

RELATION BETWEEN REQUIRED PAVEMENT WIDTH FOR TRUCKS AND FOR PASSENGER CARS

The results show that free-moving vehicles, regardless of the width of the pavement, travel closer to the center line than to the edge and, except when traffic volumes are large, there is always a definite difference between the position of free-moving vehicles and that of meeting vehicles. The body and edge clearances for meeting vehicles or perhaps for passing vehicles are, therefore, the critical factors that determine adequate pavement width.

The relation between the position of meeting vehicles and the pavement width on straight, level sections of highway is shown in figure 11. The curve for commercial vehicles meeting other commercial vehicles shows that they travel closer to the center of the lane on 18-foot than on 20-foot pavements. The drivers have no other choice if they wish to maintain some edge clearance. On 24-foot pavements they travel in the center of the lane, apparently satisfied with both edge and center clearance.

If it is assumed that the pavement is of adequate width when meeting vehicles travel in the centers of

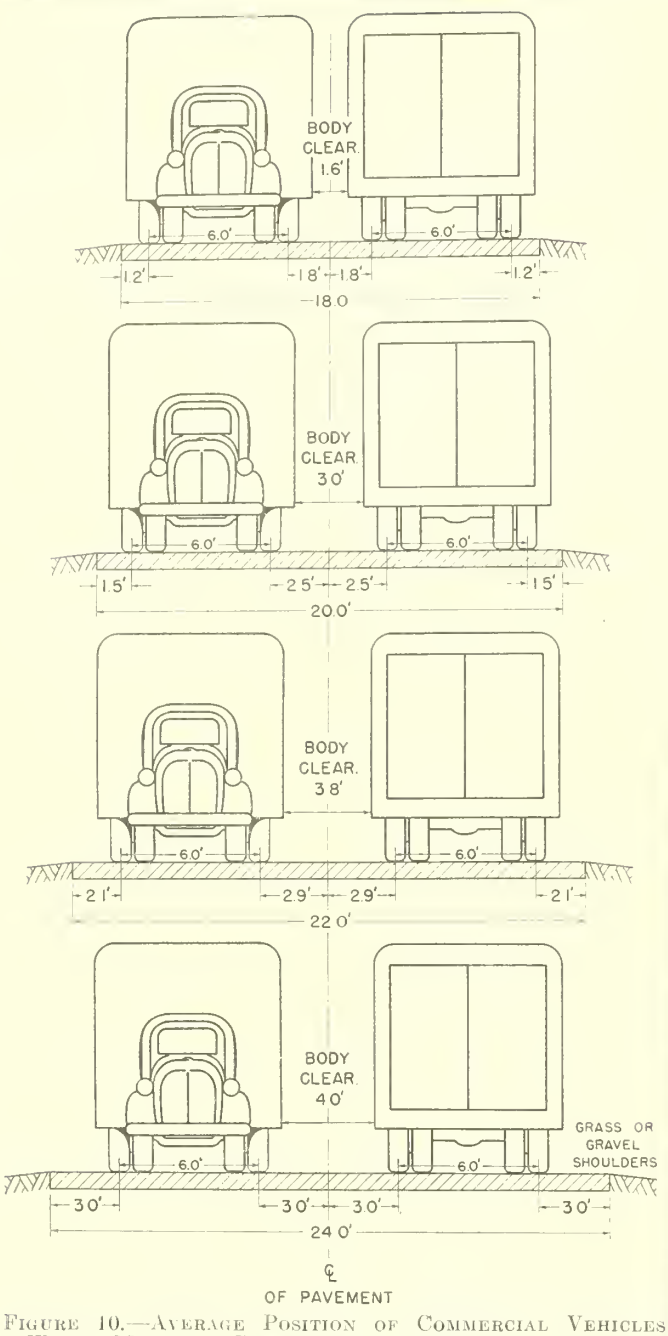


FIGURE 10.—AVERAGE POSITION OF COMMERCIAL VEHICLES WHEN MEETING COMMERCIAL VEHICLES ON TWO-LANE CONCRETE PAVEMENTS DURING THE DAY.

their respective lanes, a 19-foot pavement is adequate during the day for traffic consisting only of passenger cars; a 23-foot pavement is adequate for the passenger cars when there is mixed traffic; and a 24-foot pavement is adequate for commercial vehicles. If provision is to be made for a majority of the drivers rather than for the average driver, wider surfaces would be needed.

From figures 7 and 8, however, it is apparent that drivers of passenger cars are not satisfied with the clearances permitted by a 19-foot pavement, when meeting other passenger cars. Instead they desire either a body clearance of about 5 feet, or a clearance between their left wheels and the center line of about 3 feet. These clearances cannot be attained until the pavement width reaches 22 feet.

If, then, it is assumed that a 22-foot pavement represents a desirable width for traffic consisting entirely of passenger cars, it can be found from figure 11 that the corresponding width for mixed traffic is 24 feet, and for traffic including a high percentage of commercial vehicles a slightly greater width is desirable.

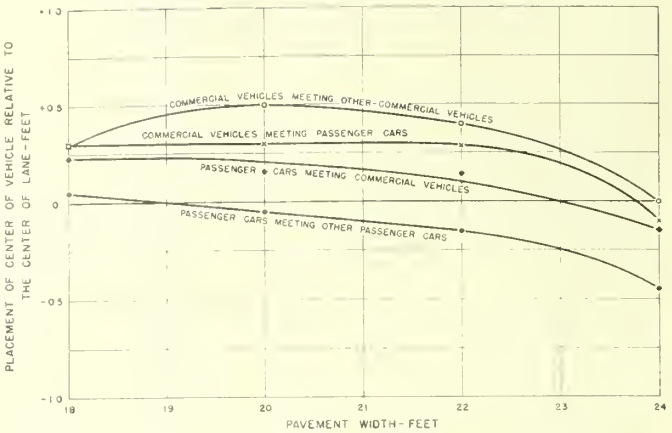


FIGURE 11.—EFFECT OF PAVEMENT WIDTH ON THE POSITION OF VEHICLES MEETING OTHER VEHICLES TRAVELING IN THE OPPOSITE DIRECTION.

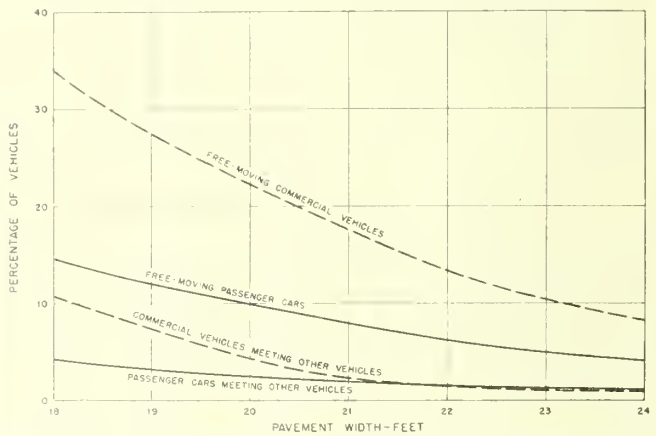
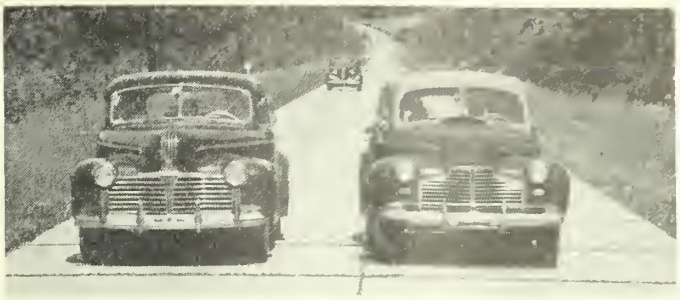


FIGURE 12.—PERCENTAGE OF VEHICLES WITH THEIR BODIES EXTENDING TO THE LEFT OF HIGHWAY CENTER LINE.

TABLE 11.—*Percentage of vehicles that travel on shoulder when meeting other vehicles during periods of light traffic*

Pavement width	Pavement with grass or gravel shoulders		Pavement with bituminous shoulders	
	Passenger cars	Commercial vehicles	Passenger cars	Commercial vehicles
	Percent	*Percent*	*Percent*	*Percent*
18 feet	0.2	5.5	4.7	17.3
20 feet	.1	1.4	1.9	6.5

On 18-foot pavements, 34 percent of the drivers of trucks and 15 percent of the drivers of passenger cars encroach on the left lane when uninfluenced by other traffic. When meeting oncoming vehicles, a surprisingly large number, 11 percent of the drivers of trucks and 5 percent of the drivers of passenger cars, fail to keep their vehicles within the proper traffic lane. These percentages decrease rapidly with an increase in pavement width. Only 1 percent of the vehicles encroach



PASSENGER CAR OVERTAKING AND PASSING OTHER PASSENGER CAR ON A 22-FOOT HIGHWAY.

on the left lane when meeting other vehicles on 24-foot pavements.

The need of pavements wider than 20 feet is further illustrated by the large increase in the percentage of vehicles that travel off the pavement as they meet other traffic when bituminous shoulders are provided (table 11).

PASSING VEHICLES DO NOT REQUIRE GREATER PAVEMENT WIDTH

Although relatively few vehicles are engaged in passing maneuvers at any one point on a two-lane highway, passing is an essential feature of two-lane highway operation. The position that the average driver of passenger cars assumes when overtaking and passing other vehicles is shown in figures 13 and 14. Included are only those passing vehicles that were definitely known to be approximately abreast of the vehicle being passed when lateral position was recorded.

Figure 13 shows the average positions of passenger cars when passing other passenger cars during the day. The average clearance between the vehicles increases from 2.3 feet on an 18-foot pavement to 4.8 feet on a 24-foot pavement. These values are from 0.7 to 1.4 feet less than the clearances between meeting passenger cars. This difference is to be expected because the difference in the speeds of passing vehicles is generally only a small fraction of the combined speeds of meeting vehicles.

On concrete pavements the driver of the passing vehicle apparently gages his transverse position by the center of the roadway. As soon as the right wheels of his vehicle are about 1.5 feet in the left lane, regardless of the pavement width, he has the clearance he desires for the maneuver. The driver of the passed car in most cases moves his vehicle laterally five-tenths foot to the right of his free-moving position.

Figure 14 shows the average position of passenger cars when passing commercial vehicles. Even though the driver of the passing vehicle moves laterally to the left more when passing commercial vehicles than when passing other passenger cars, the body clearances are smaller when commercial vehicles are passed. The clearance between a passenger vehicle passing a commercial vehicle is also less than when these vehicles meet one another traveling in opposite directions on pavements of the same width.

It may be concluded that on two-lane highways, pavement widths adequate to accommodate vehicles as they meet opposing traffic are more than adequate for passing maneuvers.

VEHICLES AVOID USE OF SHOULDERS AT NIGHT

At night, free-moving passenger cars travel farther from the edge of the pavement than in the daytime

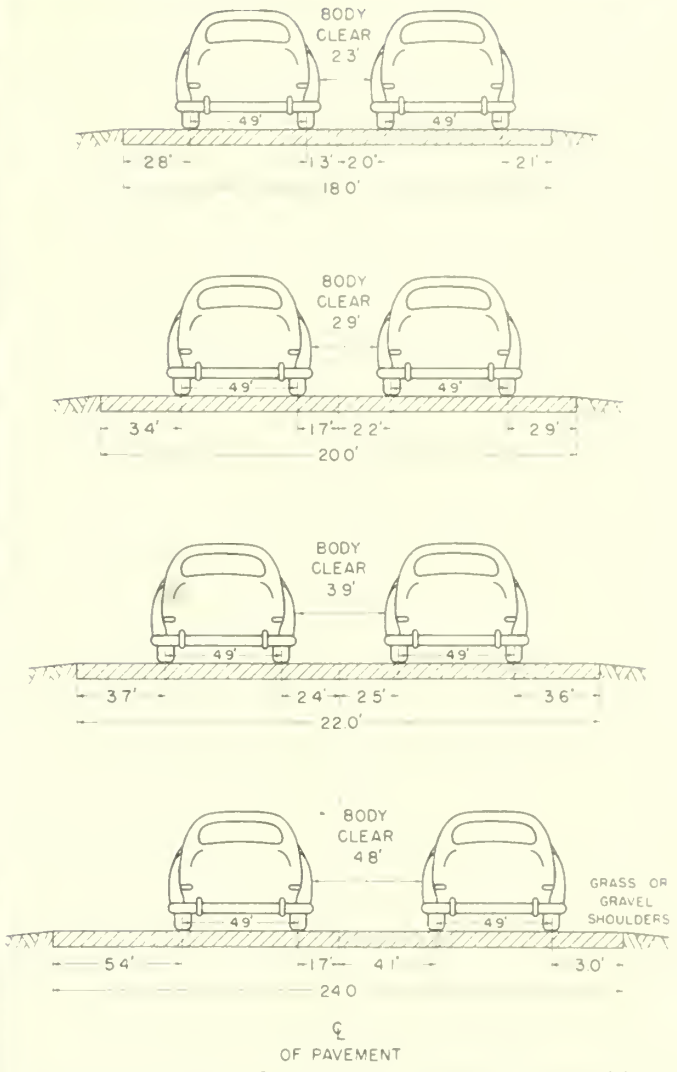


FIGURE 13.—AVERAGE POSITION OF PASSENGER CARS WHEN PASSING PASSENGER CARS ON TWO-LANE CONCRETE PAVEMENTS DURING THE DAY.

(table 6). When passenger cars meet other passenger cars at night, however, their transverse positions are practically the same as during the day. Drivers of commercial vehicles apparently prefer greater edge clearance at night than during the day for most of the several traffic conditions. Consequently, clearances between bodies of commercial vehicles when meeting other vehicles are slightly lower at night than during the day. Also, the percentage of vehicles that encroach on the left lane, both when free-moving and when meeting other vehicles, is higher at night, especially on the narrower pavements. For example, on a percentage basis, three times as many commercial vehicles encroach on the left lane when meeting other vehicles on 18-foot pavements at night as during the day.

Shoulders are used very little at night except on 18-foot pavements. On pavements of this width about 2 percent of the vehicles use the shoulders. Even bituminous shoulders are not used to a great extent at night, although they increase the effective pavement width to the same extent as during the day.

The average body clearance between passing and passed vehicles is greater for passenger cars at night than in the daytime, primarily because the driver of the

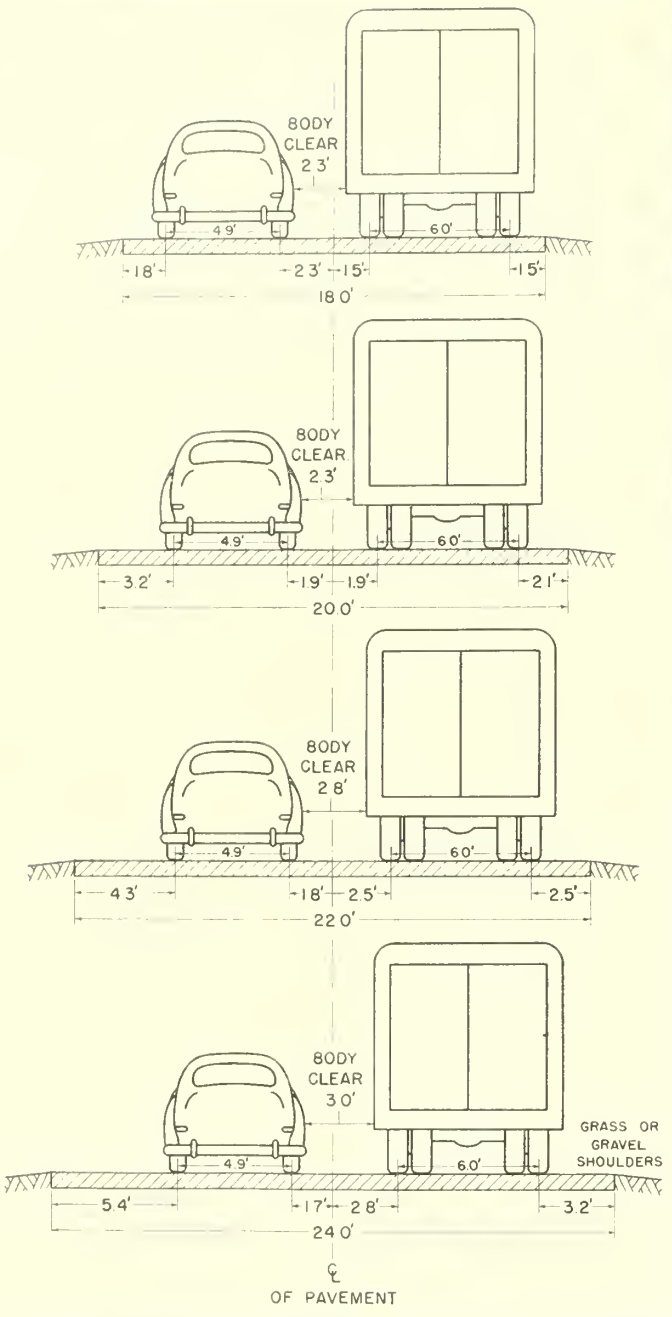


FIGURE 14.—AVERAGE POSITION OF PASSENGER CARS WHEN PASSING COMMERCIAL VEHICLES ON TWO-LANE CONCRETE PAVEMENT DURING THE DAY.

passing vehicle swerves more to the left at night. There were too few times when commercial vehicles passed at night to obtain an adequate sample (table 7).

The only definite conclusion regarding necessary surface widths for night driving is that 18-foot pavements are more inadequate at night than during the day for similar traffic densities. The wider surfaces appear to be as adequate at night as during the day.

PATHS OF PASSING VEHICLES DETERMINED BY ANALYSIS

During the speed-placement studies at each site, the transverse positions of the vehicles were recorded at only one point on the highway. The path of each vehicle, as it overtook and passed a slower-moving vehicle, was not recorded. It is possible, however, to determine the path of the average vehicle as it overtakes and passes

PASSENGER CAR OVERTAKING AND PASSING TRUCK ON A 22-FOOT HIGHWAY.

other vehicles traveling at various speeds by correlating the lateral and longitudinal positions of a large number of different vehicles with the relative positions of the vehicles being overtaken and passed. In this analysis only data for those vehicles uninfluenced by oncoming traffic were used. The combined results for four locations on 20-foot pavements in California and Texas are shown in figures 15, 16, and 17.

Figure 15 shows time-position curves that indicate the paths of passing vehicles with respect to the passed vehicle for three speed conditions. The abscissas indicate the time spacing of the two vehicles. Plus values denote that the faster car is behind the vehicle being passed and the minus values indicate the faster vehicle is ahead and is returning to its own lane. The ordinates indicate the spacing of the left wheels of the faster passenger cars from the center of the pavement.

Curve C shows the position of the left wheels of passenger cars traveling at the same speed as the vehicle ahead. The average speed was observed to be 41 miles per hour. Drivers of these cars follow the same path as the preceding vehicle if the time spacing between vehicles exceeds 2 seconds. When the spacing falls below 2 seconds, that is, when the distance between centers of cars averages 120 feet, the drivers of the trailing cars begin to edge toward the center of the pavement. When immediately behind, the rear passenger car is about 1 foot closer to the center of the road than the vehicle ahead. Included in this group are cars that seek the opportunity to pass, but are trailing at the same speed. As soon as conditions are favorable they increase their speed for a passing maneuver. The positions of vehicles that travel at speeds higher than the vehicle being passed are shown in curves A and B.

Curve B shows the path of the left wheels of cars traveling 4 to 9 miles per hour faster than the vehicle being overtaken. The average speed of the passing car and the difference in speeds of passing and passed vehicles are 46.2 and 6.4 miles per hour, respectively. The passing cars travel somewhat closer to the center of the pavement than those in curve C, and begin to veer toward the center at a time spacing of 3 seconds, or when the average distance between centers of cars is about 200 feet. The faster cars encroach on the left lane when the time spacing is 1.5 seconds, and return to the right lane after overtaking the slower vehicle when the time spacing is 1.4 seconds. However, these cars assume their original transverse position on the pavement only when they are 5 seconds ahead of the vehicle passed. When these vehicles are abreast of the vehicle being passed, their left wheels are 3.8 feet from the left pavement edge, and the body clearance is, on the average, 3 feet.

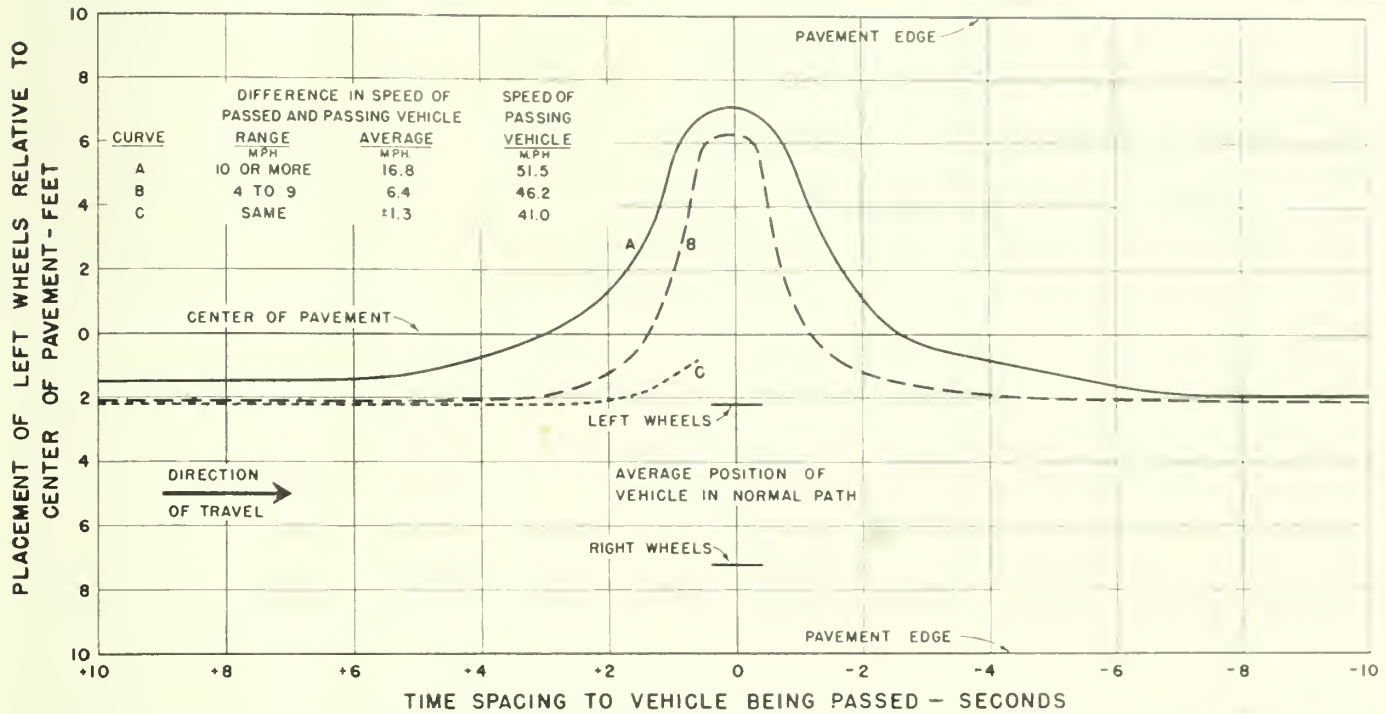

PLACEMENT OF LEFT WHEELS RELATIVE TO CENTER OF PAVEMENT - FEET
TIME SPACING TO VEHICLE BEING PASSED — SECONDS
PAVEMENT EDGE
DIFFERENCE IN SPEED OF PASSED AND PASSING VEHICLE
SPEED OF PASSING VEHICLE
CURVE
RANGE MPH
AVERAGE MPH
MPH
A 10 OR MORE 16.8 51.5
B 4 TO 9 6.4 46.2
C SAME ±1.3 41.0
A
B
C
CENTER OF PAVEMENT
LEFT WHEELS
DIRECTION OF TRAVEL
AVERAGE POSITION OF VEHICLE IN NORMAL PATH
RIGHT WHEELS
PAVEMENT EDGE
10 8 6 4 2 0 2 4 6 8 10
+10 +8 +6 +4 +2 0 -2 -4 -6 -8 -10

FIGURE 15.—EFFECT OF TIME INTERVAL BETWEEN SUCCESSIVE VEHICLES ON THE TRANSVERSE POSITION OF PASSENGER CARS TRAVELING ON 20-FOOT CONCRETE PAVEMENT. THERE WAS NO INTERFERENCE FROM APPROACHING TRAFFIC.

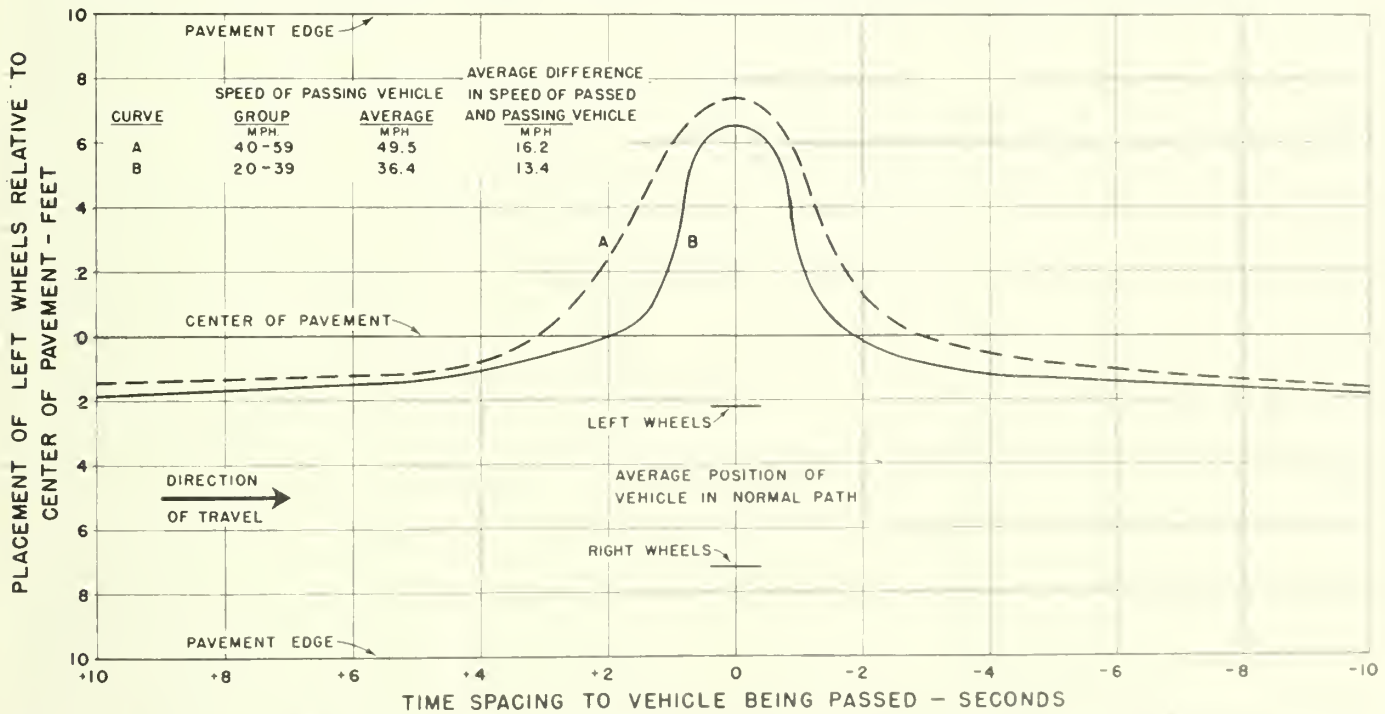

PLACEMENT OF LEFT WHEELS RELATIVE TO CENTER OF PAVEMENT - FEET
TIME SPACING TO VEHICLE BEING PASSED — SECONDS
PAVEMENT EDGE
AVERAGE DIFFERENCE IN SPEED OF PASSED AND PASSING VEHICLE
SPEED OF PASSING VEHICLE
CURVE
GROUP MPH
AVERAGE MPH
MPH
A 40-59 49.5 16.2
B 20-39 36.4 13.4
A
B
CENTER OF PAVEMENT
LEFT WHEELS
DIRECTION OF TRAVEL
AVERAGE POSITION OF VEHICLE IN NORMAL PATH
RIGHT WHEELS
PAVEMENT EDGE
10 8 6 4 2 0 2 4 6 8 10
+10 +8 +6 +4 +2 0 -2 -4 -6 -8 -10

FIGURE 16.—EFFECT OF TIME INTERVAL BETWEEN SUCCESSIVE VEHICLES ON TRANSVERSE POSITION OF PASSENGER CARS TRAVELING 10 OR MORE MILES PER HOUR FASTER THAN VEHICLE BEING PASSED ON 20-FOOT CONCRETE PAVEMENT. THERE WAS NO INTERFERENCE FROM APPROACHING TRAFFIC.

Curve A shows the path of the left wheel of cars traveling 10 or more miles per hour faster than the vehicles being overtaken. The passing cars are driven at an average speed of 51.5 miles per hour, and 16.8 miles per hour faster than the vehicle being passed. These vehicles travel much closer to the center of the pavement and move to the left lane much sooner than the vehicles in the lower speed differential groups. At a time spacing of 5 seconds, or when about 375 feet behind the slower vehicle, they begin a gradual shift toward the left lane, encroaching on the left lane at a time spacing of 3 seconds. When abreast of the vehicle being overtaken, their left wheels travel at a distance of 2.9 feet from the left pavement edge, allowing a clearance of 3.4 feet between the bodies of the two vehicles. The passing cars return to the right lane when 2.6 seconds ahead of the slower vehicle.

The curves in figure 15 are representative of the paths of vehicles that maintain a constant speed during the entire passing maneuver. Under actual operating

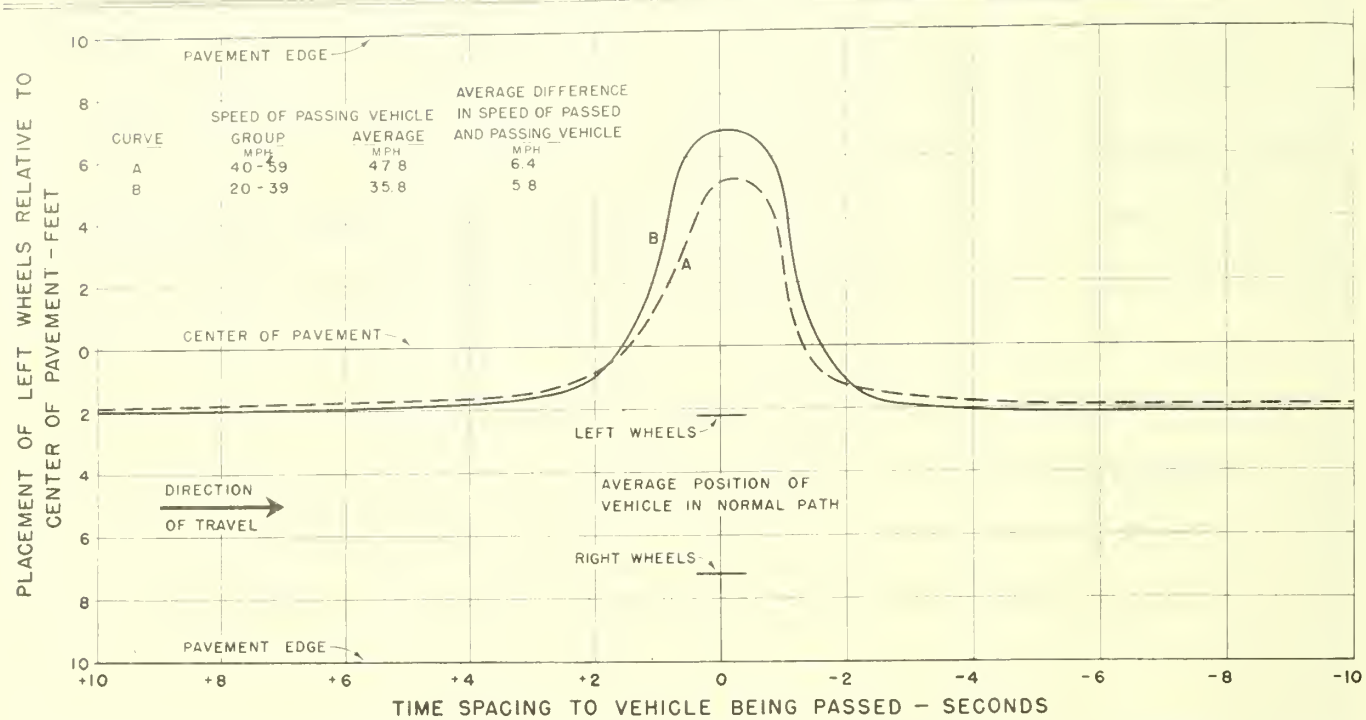

PAVEMENT EDGE
SPEED OF PASSING VEHICLE
AVERAGE DIFFERENCE IN SPEED OF PASSED AND PASSING VEHICLE
CURVE
GROUP MPH
AVERAGE MPH
MPH
A 40-59 47.8 6.4
B 20-39 35.8 5.8
CENTER OF PAVEMENT
LEFT WHEELS
AVERAGE POSITION OF VEHICLE IN NORMAL PATH
RIGHT WHEELS
DIRECTION OF TRAVEL
PAVEMENT EDGE
PLACEMENT OF LEFT WHEELS RELATIVE TO CENTER OF PAVEMENT - FEET
TIME SPACING TO VEHICLE BEING PASSED - SECONDS
+10 +8 +6 +4 +2 0 -2 -4 -6 -8 -10

FIGURE 17.—EFFECT OF TIME INTERVAL BETWEEN SUCCESSIVE VEHICLES ON TRANSVERSE POSITION OF PASSENGER CARS TRAVELING 4 TO 9 MILES PER HOUR FASTER THAN VEHICLE BEING PASSED ON 20-FOOT CONCRETE PAVEMENT. THERE WAS NO INTERFERENCE FROM APPROACHING TRAFFIC.

conditions some passing maneuvers are performed in this manner, but generally the driver of the passing vehicle accelerates as he encroaches on the left lane. The actual path of the passing vehicle that accelerates would be governed by the time spacing relative to the vehicle being passed when the maneuver is started, by the speeds of the vehicles, and by the rate of acceleration. For example, as a vehicle traveling at the same speed as the vehicle to be passed accelerates, its path might shift from curve C to curve B and then from curve B to curve A before getting abreast of the passed vehicle.

In figure 15, the controlling factor for each of the curves is the speed differential between the passed and passing vehicles. Data for passing vehicles traveling at all speeds are combined. Figure 16 shows the effect of speed on the paths of the passing vehicles when there is a speed differential of at least 10 miles per hour. Figure 17 shows the effect of speed when there is a speed differential between 4 and 9 miles per hour. These figures show that for the same speed differential, the portion of the pavement width that is utilized varies with the speed of the passing vehicles.

DISTANCE COMPUTED FOR OCCUPATION OF LEFT LANE BY PASSING VEHICLES

Although figures 15 through 17 show the relative transverse positions of vehicles with various time spacings on the highway, they do not show the distance that the faster vehicles travel in the left lane. This distance can easily be calculated from the curves. For example, curve A of figure 15 shows that the faster car while traveling in the left lane gains 5.6 seconds on the vehicle being passed, 3 seconds of which are gained from the time the passing vehicle encroaches on the left lane until it is abreast of the passed vehicle, and 2.6 seconds are gained while returning to the right lane. At 51.5 miles per hour a gain of 5.6 seconds represents a gain of $5.6 \times 51.5 \times 1.47$, or 423 feet. In other words, the faster car is 227 feet behind when it enters the left lane to pass the slower moving vehicle, and 196 feet ahead when it returns to the right lane. The time spent by the faster vehicle in the left lane equals the distance divided by the difference in speed. Thus, time spent in left lane is 423 divided by 16.8×1.47, or 17.2 seconds. During this elapsed time, the faster car travels 1,299 feet in the left lane, while the slower vehicle travels 876 feet. Similarly, curve B shows that a passenger car traveling 4 to 9 miles per hour faster than the vehicle being overtaken gains 2.7 seconds and travels at an average speed of 46.2 miles per hour. The faster car travels 1,322 feet in the left lane while the slower vehicle travels 1,139 feet.

Table 12 shows the speed and transverse position of the passing vehicle and the time and distance traveled by passenger cars in the left lane while overtaking slower vehicles on two-lane concrete pavement 20 feet in width.

This analysis of vehicle paths during passing maneuvers supplies information that cannot be obtained by any other practical procedure. The results augment those of the passing studies in which more comprehensve data regarding the travel time in the left lane and speeds at various stages of the passing maneuvers were recorded and analyzed.[4]

STUDIES RESULT IN SIGNIFICANT CONCLUSIONS

Based upon the results of extensive studies of vehicle operation at a large number of highway locations, the following are the more important conclusions that apply to straight, level sections of two-lane highway with concrete surfaces:

1. Shoulder width in excess of 4 feet does not influence the effective pavement width for moving vehicles

[4] Procedure Employed in Analyzing Passing Practices of Motor Vehicles, by E. H. Holmes, PUBLIC ROADS, January 1939.
Progress in Study of Motor-Vehicle Passing Practices, by O. K. Normann, PUBLIC ROADS, February 1940.
Passing Practices on Rural Highways, by C. W. Prisk, Highway Research Board Proceedings, 1941.

TABLE 12.—*Speed, placement, and distance traveled in left lane by passenger cars while overtaking slower vehicles on two-lane concrete pavements 20 feet wide*

CARS TRAVELING 4 TO 9 MILES PER HOUR FASTER THAN VEHICLES BEING OVERTAKEN

Speed		Average difference in speed	Travel in left lane		Distance gained while in left lane	Transverse [1] position when abreast of slower vehicle	Body clearance when abreast of slower vehicle
Range	Average		Time	Distance			
Miles per hour	*Miles per hour*	*Miles per hour*	*Seconds*	*Feet*	*Feet*	*Feet*	*Feet*
20–39	35.8	5.8	19.8	1,040	168	3.1	3.2
40–59	47.8	6.4	20.1	1,409	189	4.7	1.6
18–72 (total)	46.2	6.4	19.5	1,322	183	3.8	2.5
CARS TRAVELING 10 OR MORE MILES PER HOUR FASTER THAN VEHICLES BEING OVERTAKEN							
20–39	36.4	13.4	10.5	561	206	3.4	2.9
40–59	49.5	16.2	18.3	1,329	436	2.6	3.7
18–72 (total)	51.5	16.8	17.2	1,299	423	2.9	3.4

[1] Distance of left wheels from left edge of pavement.

when there are no vertical obstructions immediately adjacent to the shoulders. This must not be interpreted that shoulders wider than 4 feet are not necessary for other important reasons.

2. Well-maintained grass shoulders have the same effect on the transverse position of moving vehicles as well-maintained gravel shoulders.

3. Bituminous-treated shoulders, 4 feet or more in width, adjacent to 18- and 20-foot pavements, increase the effective surface width approximately 2 feet.

4. Lip curbs on 20-foot pavements reduce the total effective pavement width during the day approximately 1 foot. At night this is true only for commercial vehicles.

5. Shoulder use increases rapidly with a decrease in pavement width below 22 feet. An insignificant number of moving vehicles use the shoulders on pavements of that width. On 18-foot pavements with grass or gravel shoulders, however, 5 percent of the commercial vehicles use the shoulder as they meet oncoming traffic. The corresponding value is 17 percent on highways with bituminous shoulders.

6. Hazardous traffic conditions exist on pavements less than 22 feet wide that carry even moderate volumes of mixed traffic. On 18-foot pavements with grass or gravel shoulders, 11 percent of the drivers of trucks and 5 percent of the drivers of passenger cars fail to keep their vehicles within their proper traffic lane even when meeting oncoming traffic.

7. Drivers do not reduce their speeds when meeting other vehicles on narrow pavements even though body clearances are definitely inadequate.

8. When a passenger car meets a commercial vehicle on a pavement 22 feet wide, the passenger car has the desired center and edge clearances but the commercial vehicle does not. To permit desired clearances for commercial vehicles a 24-foot pavement is required.

APPENDIX

During the analysis of the data from field studies, a large number of tables and graphs were prepared showing frequency distributions, mean and standard deviations, and other statistical measures pertaining to the variation in speeds and transverse placements of vehicles under various conditions. These tables and graphs were used in arriving at the conclusions but are omitted from the report the discussion being based primarily on average values. It is believed, however, that the frequency distributions of vehicle placements may be useful in determining slab thickness and cross section, shoulder width and type, and other design features, in addition to their value in determining required surface width for a specific traffic condition.

TRAFFIC ON TWO-LANE HIGHWAY. NOTE THE TENDENCY OF DRIVERS TO TRAVEL CLOSER TO THE CENTER OF THE HIGHWAY WHEN TRAILING OTHER VEHICLES. THE FIRST THREE LINES ACROSS PAVEMENT ARE SPEED AND PLACEMENT DETECTORS. DETECTORS ARE CONNECTED TO INSTRUMENTS NOT SHOWN IN THE PICTURE.

Table 1 of the appendix shows frequency distributions of transverse placements for the more important vehicle classifications on two-lane concrete pavements 18, 20, 22, and 24 feet wide with grass or gravel shoulders for day operations. For night operations typical distributions are shown for 18- and 20-foot pavement widths.

Table 2 shows the frequency distribution of clearances between bodies of vehicles as they met one another traveling in opposite directions.

TABLE 1.—*Frequency distribution of transverse placements of vehicles on two-lane concrete pavements with grass or gravel shoulders at least 4 feet in width*

18-FOOT PAVEMENTS DURING DAY

Placement of left wheels to the right of center line [1]		Passenger cars				Commercial vehicles		
Range (feet)	Average	Free moving	Meeting other passenger cars	Meeting commercial vehicles	All, except passing vehicles	Free moving	Meeting passenger cars	All, except passing vehicles
	Ft.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
−3.9 to −3	−3.5	0.1			(2)			
−2.9 to −2	−2.5	.2	(2)		0.2	0.7	0.7	0.7
−1.9 to −1	−1.5	1.2	0.5	0.1	.9	.8	.7	.5
−.9 to 0	−.5	4.2	.4	1.2	3.4	2.6	2.5	2.5
0 to .9	.5	21.6	5.0	3.5	16.2	29.9	6.8	21.3
1 to 1.9	1.5	19.5	32.0	24.5	45.9	18.5	51.1	49.1
2 to 2.9	2.5	21.4	53.8	52.1	29.7	16.7	27.8	21.2
3 to 3.9	3.5	1.7	8.1	18.6	3.5	.8	7.4	1.8
4 to 4.9	4.5	.1	.2		.1			0
5 to 5.9	5.5				0			.1
6 to 6.9	6.5				.1			
Total		100	100	100	100	100	100	100
Average distance, feet		1.4	2.1	2.3	1.6	1.3	1.8	1.4

20-FOOT PAVEMENTS DURING DAY

−3.9 to −3	−3.5	0.1			0.1			
−2.9 to −2	−2.5	.3	(2)		.2	0.1	0.3	0.3
−1.9 to −1	−1.5	1.5	0.1	0.3	1.2	.6	.5	.5
−.9 to 0	−.5	4.6	1.0	.7	3.5	1.3	.4	2.1
0 to .9	.5	17.4	3.1	3.0	11.0	17.2	3.2	14.1
1 to 1.9	1.5	41.3	19.1	17.5	35.5	42.2	27.2	37.7
2 to 2.9	2.5	28.6	46.2	43.0	31.4	30.8	46.4	35.9
3 to 3.9	3.5	5.4	26.8	33.4	10.0	3.8	20.6	7.1
4 to 4.9	4.5	.6	3.7	2.1	1.0	.8	1.1	.9
5 to 5.9	5.5	1	(2)		(2)	2		(2)
6 to 6.9	6.5	1	(2)		1	1		(2)
Total		100	100	100	100	100	100	100
Average distance, feet		1.6	2.5	2.7	1.8	1.1	2.0	1.4

See footnotes at end of table.

TABLE 1.—*Frequency distribution of transverse placements of vehicles on two-lane concrete pavements with grass or gravel shoulders at least 4 feet in width*—Continued.

22-FOOT PAVEMENTS DURING DAY

Placement of left wheels to the right of center line[1]		Passenger cars				Commercial vehicles		
Range (feet)	Average	Free moving	Meeting other passenger cars	Meeting commercial vehicles	All, except passing vehicles	Free moving	Meeting passenger cars	All, except passing vehicles
	Ft.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
−2.9 to −2	−2.5	0.2	.....	.....	0.2	.....	.....	.....
−1.9 to −1	−1.5	1.2	0.8	.....	.9	1.4	.....	1.5
−.9 to 0	−.5	1.8	0	.....	1.4	5.3	.....	4.9
0 to .9	.5	13.5	4.0	.....	9.9	7.9	1.4	7.5
1 to 1.9	1.5	25.4	8.9	17.4	21.6	22.8	19.0	21.1
2 to 2.9	2.5	40.6	34.9	30.5	40.1	47.7	40.1	47.3
3 to 3.9	3.5	13.9	39.1	26.1	20.5	13.5	23.8	15.4
4 to 4.9	4.5	2.8	11.0	17.4	4.7	1.4	15.7	2.3
5 to 5.9	5.5	.6	1.3	4.3	.6	.....	.....	.....
6 to 6.9	6.5	.....	.....	4.3	.1	.....	.....	.....
Total	.....	100	100	100	100	100	100	100
Average distance, feet	.....	2.1	2.9	3.2	2.3	2.0	2.8	2.1

24-FOOT PAVEMENTS DURING DAY

−2.9 to −2	−2.5	0.3	.....	.....	0.2	0.6	.....	0.3
−1.9 to −1	−1.5	.2	0.3	.....	.3	.6	.....	.5
−.9 to 0	−.5	1.3	1.3	.....	1.1	2.3	.....	1.2
0 to .9	.5	9.0	2.9	.....	5.6	4.6	1.2	3.1
1 to 1.9	1.5	24.8	7.7	4.2	18.1	33.7	21.6	27.4
2 to 2.9	2.5	33.5	31.0	35.4	33.6	37.1	37.1	39.4
3 to 3.9	3.5	24.0	34.8	33.3	27.4	17.1	21.6	20.3
4 to 4.9	4.5	5.8	19.8	25.0	12.1	3.4	17.0	6.9
5 to 5.9	5.5	1.1	2.2	2.1	1.6	.6	1.5	.9
Total	.....	100	100	100	100	100	100	100
Average distance, feet	.....	2.4	3.1	3.4	2.7	2.2	2.9	2.5

18-FOOT PAVEMENTS AT NIGHT

−3.9 to −3	−3.5	(2)	.....	.....	.....	.....	.....	.....
−2.9 to −2	−2.5	0.1	0.2	.....	0.2	0.2	.....	0.2
−1.9 to −1	−1.5	.9	.8	.....	.8	3.6	.....	.9
−.9 to 0	−.5	6.0	.9	.....	5.3	10.5	6.7	8.5
0 to .9	.5	21.8	5.7	.....	18.5	41.8	20.0	34.3
1 to 1.9	1.5	45.9	31.9	16.7	44.0	32.7	53.3	40.7
2 to 2.9	2.5	22.3	48.1	66.7	26.9	9.0	13.3	14.2
3 to 3.9	3.5	1.9	12.2	16.6	3.3	2.2	6.7	1.2
4 to 4.9	4.5	.4	.2	.....	.4	.....	.....	.....
5 to 5.9	5.5	0	.....	.....	0	.....	.....	.....
6 to 6.9	6.5	.7	.....	.....	.6	.....	.....	.....
Total	.....	100	100	100	100	100	100	100
Average distance, feet	.....	1.5	2.1	2.5	1.5	0.9	1.4	1.2

20-FOOT PAVEMENTS AT NIGHT

−3.9 to −3	−3.5	(2)	.....	.....	0.1	0.2	.....	(2)
−2.9 to −2	−2.5	0.9	0.2	.....	.2	.5	.....	0.3
−1.9 to −1	−1.5	2.5	.2	.....	1.2	.5	1.1	.7
−.9 to 0	−.5	5.8	1.2	4.5	3.5	5.0	2.0	2.7
0 to .9	.5	18.4	2.2	4.5	14.0	23.9	16.3	14.1
1 to 1.9	1.5	38.6	18.4	8.6	35.5	42.7	33.1	37.7
2 to 2.9	2.5	27.8	47.3	44.3	34.4	22.3	44.2	35.9
3 to 3.9	3.5	5.0	28.1	34.8	10.0	4.9	3.0	7.7
4 to 4.9	4.5	.8	2.4	3.3	1.0	.....	.....	.9
5 to 5.9	5.5	.1	.....	.....	(2)	.....	.....	.....
6 to 6.9	6.5	.1	.....	.....	.1	.....	.....	.....
Total	.....	100	100	100	100	100	100	100
Average distance, feet	.....	1.6	2.5	2.7	1.8	1.5	1.9	1.8

[1] A minus value indicates that left wheel is to left of center line.
[2] Less than 0.05 percent.

TABLE 2.—*Frequency distribution of clearances between bodies of meeting vehicles on two lane concrete pavements with grass or gravel shoulders at least 4 feet in width*

PASSENGER CARS MEETING OTHER PASSENGER CARS

Clearance between vehicle bodies		Day				Night	
Range (feet)	Average	18-foot pavement	20-foot pavement	22-foot pavement	24-foot pavement	18-foot pavement	20-foot pavement
	Ft.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
0 to 2	1	6.1	3.8	2.6	2.4	4.3	3.8
1 to 3	2	16.2	7.6	5.2	1.4	16.0	5.3
2 to 4	3	41.7	19.5	11.1	5.8	36.1	21.7
3 to 5	4	29.3	36.4	15.3	15.5	29.9	37.9
4 to 6	5	6.5	22.9	37.5	29.5	13.3	25.1
5 to 7	6	.2	6.6	19.4	31.8	.4	5.4
6 to 8	7	.....	2.6	8.1	12.6	.....	.8
7 to 9	8	.....	.6	.8	1.0	.....	.....
8 to 10	9	.....	(1)	.....	.....	.....	.....
Total	.....	100	100	100	100	100	100
Average clearance, feet	.....	3.2	4.0	4.8	5.2	3.2	4.0

PASSENGER CARS AND COMMERCIAL VEHICLES MEETING

0 to 1.5	0.8	7.3	2.5	2.3	1.5	6.1	13.9
0.5 to 2.5	1.5	14.8	3.8	4.5	1.7	15.1	4.0
1.5 to 3.5	2.5	50.2	24.7	6.8	9.2	57.6	20.9
2.5 to 4.5	3.5	23.6	31.6	13.6	16.6	12.1	29.4
3.5 to 5.5	4.5	3.4	26.4	38.7	24.1	6.1	29.0
4.5 to 6.5	5.5	.7	9.4	25.0	29.8	3.0	2.8
5.5 to 7.5	6.5	.....	1.3	6.8	14.2	.....	.....
6.5 to 8.5	7.5	.....	.3	2.3	2.7	.....	.....
7.5 to 9.5	8.5	.....	.....	.....	.2	.....	.....
Total	.....	100	100	100	100	100	100
Average clearance, feet	.....	2.6	3.5	4.5	4.8	2.6	3.1

[1] Less than 0.05 percent.

TRAFFIC PLANNING STUDIES IN AMERICAN CITIES

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by JOHN T. LYNCH, Chief of Planning Surveys Section

DURING THE PAST 2 YEARS there has been a remarkable development in city planning studies. Techniques have been greatly improved and many new studies have been begun.

The principal reason for this accelerated activity is the provision of the Federal-Aid Highway Act of 1944, which makes available $125,000,000 of Federal money in each of the first 3 years following the war for construction on the Federal-aid system in urban areas. Since this money is to be matched by State and local funds, the total sums will be about twice that amount.

NEW FEDERAL POLICY

When distributed among the States and among the cities in the States, this is not a large amount of money considering the magnitude of the problems to be solved. It does, however, mark a change in Federal policy which may result in the availability of considerable sums of money for urban construction over the next 20-year period. It was not until the emergency legislation of 1933 that Federal highway funds could legally be spent for construction through municipalities having a population of more than 2,500 where the houses average less than 200 feet apart.

Since enactment of the Hayden-Cartwright Act of 1934 regular Federal-aid funds have been available for expenditure on urban sections of the Federal-aid system and numerous improvements have been made. However pressing needs for improvements on rural portions of the system and the size of the funds provided have prevented a major attack on city highway problems. The new legislation provides $225,000,000 for either urban or rural portions of the Federal-aid system in each of the first 3 postwar years. It definitely recognizes the urban highway problem as, in part, a Federal responsibility by providing an additional $125,000,000 in each of the first 3 postwar years that is specifically assigned to urban portions of the Federal-aid system.

The legislation provides for designation and improvement of a 40,000-mile system of interstate highways with appropriate connections through cities. This system is to be made a part of the Federal-aid system and will be eligible for participation in Federal-aid system funds. No special funds have been provided for the interstate system.

Funds provided by the new legislation and general recognition of the necessity of solving the more critical city traffic problems by large scale, bold, and costly improvements assure large city programs. The concept of the scale and character of these programs is relatively new. Decisions made now as to the pattern for improvements will influence developments for many years to come. It is especially important that data be obtained and analyzed on an extensive scale in such manner as to clarify the complex urban problems and indicate the proper solutions. Recognizing that the methods used in rural-road analysis and those generally used in city planning in the past are not adequate for the solution of problems now at hand, the Public Roads Administration has made special studies of methods of accumulating the data needed for scientific planning of urban highways.

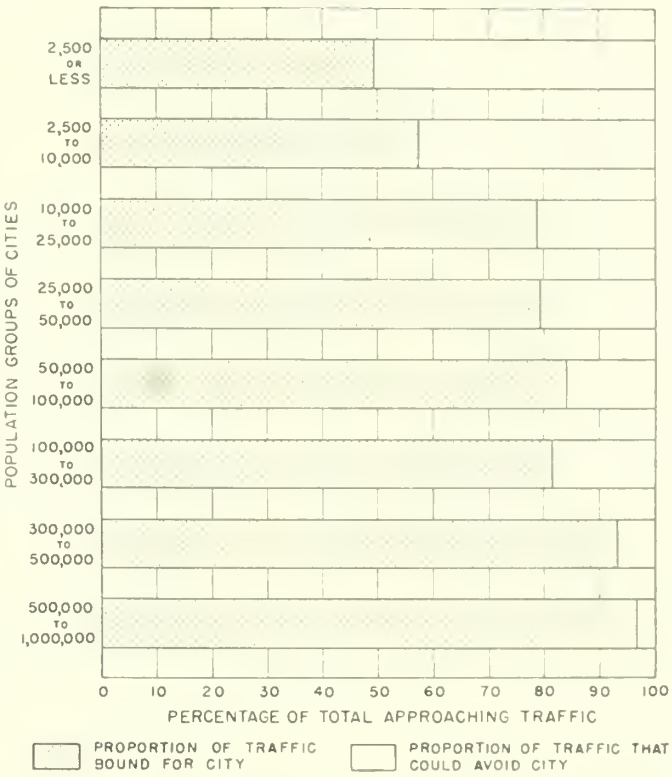


FIGURE 1.—GRAPH SHOWING TWO DIVISIONS OF THE TOTAL TRAFFIC ON ROADS APPROACHING REPRESENTATIVE CITIES OF VARIOUS POPULATION GROUPS: (1) THE AVERAGE PERCENTAGE CITY BOUND AND (2) THE AVERAGE PERCENTAGE WHICH COULD HAVE BYPASSED THE CITIES.

BYPASS STUDIES

Having thought in terms of rural roads for so many years, the inclination of the State and Federal engineers on first approaching city problems was to regard urban congestion as a hindrance to the flow of intercity and interstate traffic and to try to expedite travel by the construction of routes skirting the edge of the urban areas. In many cases such bypasses have been justified, but early studies showed that most of the traffic approaching a city wants to go into it and that a bypass solves only a small part of the problem.

Figure 1, taken from the report of the National Interregional Highway Committee which was transmitted to Congress before the passage of the Federal Highway Act of 1944, shows the percentage of the traffic approaching cities of various sizes which is bound for the city, and the percentage which could avoid the city. It must be emphasized that these are averages and that the percentages vary widely according to the conditions. It will be noted that for cities of over 500,000 population about 96 percent of the

FIGURE 2.—PRINCIPAL HIGHWAY ROUTES THROUGH MARTINSVILLE, VA.

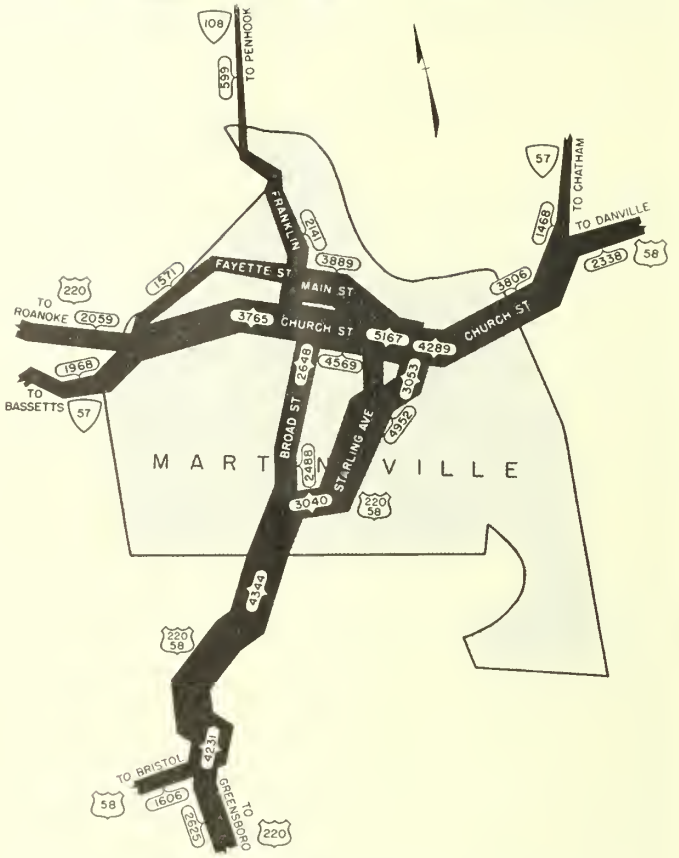
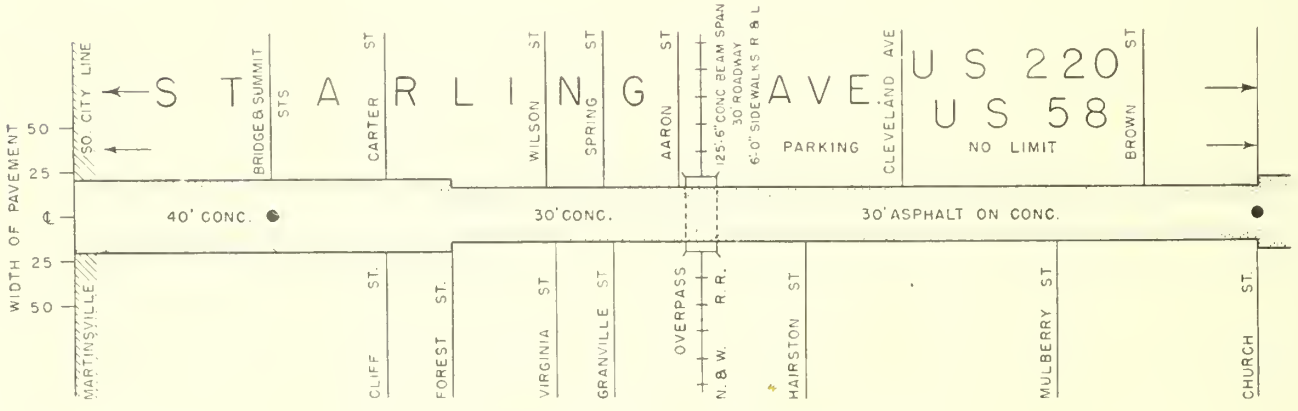
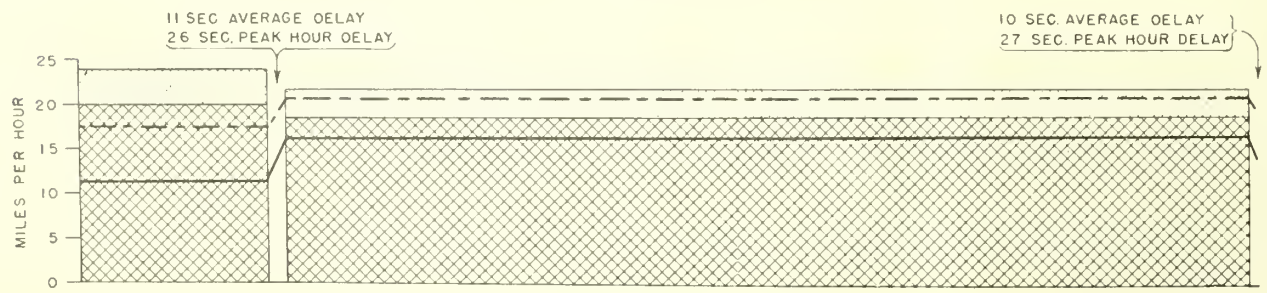


FIGURE 3.—ESTIMATED AVERAGE 24-HOUR TRAFFIC FLOW ON MAJOR STREETS IN MARTINSVILLE, VA., IN 1941.



WIDTH AND TYPE OF PAVEMENT



AVERAGE SPEED

FIGURE 4.—WIDTH AND TYPE OF PAVEMENT, AVERAGE SPEED, AND

approaching traffic is bound for the city, whereas for cities of from 10,000 to 25,000 the corresponding figure is about 79 percent, on the average.

Smaller cities frequently need bypasses because they will contribute materially to the relief of traffic congestion within the city. To illustrate the kind of studies being made to determine the need for bypasses, a survey made in Martinsville, Va., by the State highway department in cooperation with the Public Roads Administration will be briefly described and some of the results will be given.

MARTINSVILLE STUDY USED AS ILLUSTRATION

Martinsville, a city of about 10,000 population, is located in southwest Virginia in the foothills of the Blue Ridge Mountains. It has an average elevation of above 1,000 feet and the surrounding topography is comparatively rough. As is usual in cities so situated, the streets have steep grades and are rather narrow. The chief industry is the manufacture of furniture, but there is diversified manufacturing including textile and other products.

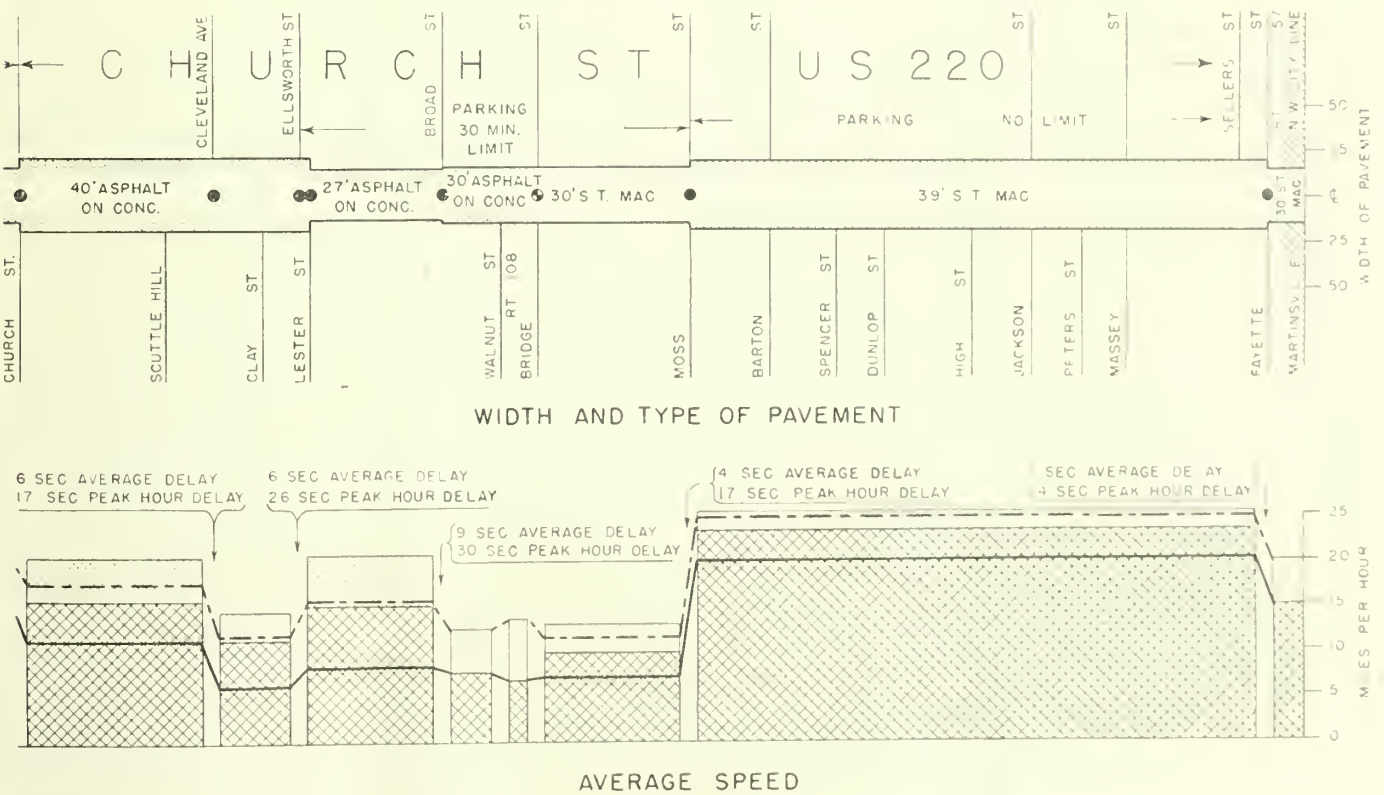
Figure 2 shows the principal highway routes through the city. U S 58 comes in from the southwest from Bristol, Va., follows Starling Avenue to Church Street, turns right on Church Street and goes out in an easterly direction to Danville, Va. U S 220 from Greensboro, N. C., joins U S 58 south of the corporate limits and is coincident with it along Starling Avenue to Church Street. At this point, however, U S 220 turns left and follows Church Street through the retail business district, and leaves the city in a westerly direction to Roanoke, Va. State route 57 coming in from the west from Bassetts, Va., joins with U S 220 near the west city limit and follows Church Street all the way through the city, coinciding with U S 58 from Starling Avenue to the east city limit, and thence turns and goes in a northerly direction to Chatham, Va. Two of the three principal routes through the city follow Church Street through the congested downtown section. A fourth route, State route 108, which is of less importance, comes in from the north and terminates in the business section at Church Street.

Figure 3 shows the traffic volumes on these routes and on other major streets in Martinsville. The greatest traffic volume on any portion of the transcity connections amounts to 5,167 vehicles per day and occurs on Church Street between Starling Avenue and Lester Street. At Lester Street this traffic divides and part of it follows Main Street.

A portion of the 4,344 vehicles coming in each day from the south leave Starling Avenue at Broad Street and another portion turns off on Cleveland Street. Both Broad and Cleveland Streets offer shorter routes to the downtown section and would be used by a much larger portion of the traffic were they equal in physical characteristics to Starling Avenue.

Figure 4 shows conditions through the city along U S 220. At the south city limit there is a 40-foot concrete pavement which narrows to 30 feet at Forest Street and the pavement continues at this width to the point where the route turns to the left from Starling Avenue on to Church Street. The first section on Church Street is 40 feet wide, but from Lester to Moss Streets, through the congested downtown section, the width is only 27 to 30 feet. Except for a very short section of 30-foot width near the west city limit, the remainder of the route is 39 feet in width.

There are numerous grades on this route, some of them rather steep. From the south corporate limit to Church Street the maximum is 5.2 percent, and on Church Street through the business section, the range is from 2.6 percent to 4.6 percent. For the half mile between Moss Street and Jackson Street there are



DELAY ON ROUTE 220 THROUGH MARTINSVILLE, VA., AS OF JULY 1944.

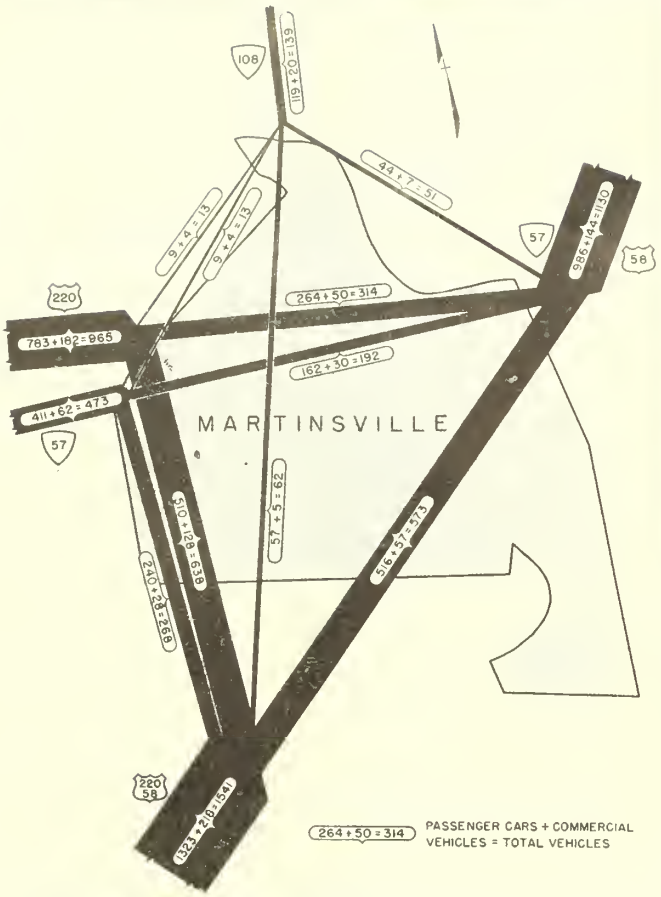


FIGURE 5.—ORIGIN AND DESTINATION OF DAILY THROUGH TRAFFIC AT MARTINSVILLE, VA., IN 1941.

continuous grades, ranging from 4 percent to 6.6 percent, and from Jackson Street to the west city limits the grades range from 4 percent up to 8 percent.

There are eight stop lights and one caution light on this route and at several other points there are interferences by cross traffic which cause considerable delay. The diagram at the bottom of the chart shows the average speeds on U S 220 through the city for all hours of the day and at the peak hour. This information was determined by cruising with the traffic. From the southern limit to Church Street conditions are fairly satisfactory, and automobiles maintain an average speed of 22 miles or more for the day, and 18 miles or more during the peak hour. After turning on to Church Street there are numerous stop lights and other interferences and average speeds decline. In the most congested section between Broad and Moss Streets the average speed for the day was from 12 to 13 miles per hour and during the rush period was from 7 to 9 miles per hour. From Moss Street to the west city limit fairly satisfactory speeds are maintained.

ORIGIN AND DESTINATION SURVEY MADE

In order to determine the improvements needed to serve through traffic, and the relation between the benefits which would accrue to the traveling public, and the cost of the improvements, "origin and destination stations" were operated on each main highway where it enters the city. Each station was manned for a full 24-hour period by interviewers who stopped all vehicles leaving the city and asked the drivers their origins and destinations and if stops were made in the city that required them to follow the route traveled even though a more attractive and more direct route were provided.

About 80 percent of the traffic had origin or destination within the city or made essential stops there, and only about 20 percent could have used a bypass, had one been available. However, conditions in this city are such that the construction of a bypass may be justified for even this small portion of the total traffic. Further analysis was made of the origins and destinations of the through traffic to determine how much of it would use the various segments of a circumferential route.

Figure 5 shows the interchange of the bypassable traffic between the major highways entering the city. The largest single through movement consists of 510 passenger cars and 128 trucks, or a total of 638 vehicles, which travel between U S 220 to the west and the coincident routes U S 220 and U S 58 to the south. The only movement of comparable magnitude is that between U S 58 and U S 220, combined, at the south and U S 58 and State route 57, combined, at the northeast.

The various types of improvements that might relieve existing conditions were considered. If existing streets could be improved in such a manner as to permit the free flow of traffic, or if other parallel facilities could be provided within the city itself, the greatest benefits could be expected because so much of the traffic has its origin or destination within the city. However, this was found to be impracticable for several reasons; Church Street is narrow and closely built up and there are no parallel streets except Main, which is already seriously congested. There is much interference by cross traffic and the total volume of traffic to be served does not justify the cost of a controlled-access highway. Even a new street at grade would require an enormous expenditure for right-of-way and would not give full relief because of the cross traffic.

POSSIBLE SOLUTIONS CONSIDERED

It was decided then to study each of the segments of a circumferential or bypass route to determine what benefits could be expected from each. A route bordering the northern part of the city would serve 506 vehicles for its entire length, and 51 additional on its eastern half, and 26 additional on its western half, the additional traffic being contributed by State route 108. At most, this would relieve Church Street of only about 10 percent of its present traffic. It is not practicable to locate the bypass so as to save distance for any except a few vehicles from State route 108, and it is probable that the route would be longer than the existing route for most of the traffic. It would be possible to save 3½ minutes in travel time for east-west traffic, but this small benefit would hardly justify the cost of construction. It is fairly obvious, without further analysis, that such a route would not be justified.

A bypass skirting the city on the east would serve 573 vehicles for its entire length, but would be as long or longer than the existing route. It would save little travel time because the existing route is fairly adequate and delays are not comparable to those encountered on the routes traversing the business section. On the whole, such a bypass would be even less justifiable than one around the northern edge of the city.

A road bypassing the city on the southwest and connecting U S 220 and U S 58 from the south with U S 220 and State route 57 to the west would obviously save

considerable distance over existing routes and, in addition, would serve the largest single movement of through traffic; namely, that which enters and leaves by U S 220. The benefits that would accrue from the construction of such a route will be analyzed in some detail. This analysis is presented for purposes of illustration and varies in some particulars from the analysis presented in a report on this project.

SOUTHWEST BYPASS ANALYZED

The approximate location of the bypass under consideration is labeled "assumed bypass" in figure 2. This location would save approximately 2 miles for through traffic on U S 220, the distances being 4.5 miles by the present route and 2.5 miles by the bypass. For traffic between U S 220 and U S 58 to the south and State route 57 to the west the saving would be 2.2 miles. Assuming that traffic will increase 60 percent during the life of the improvement and will average 30 percent higher during that period than at present, the average daily vehicle-miles saved would be as follows:

Passenger cars	2,012
Trucks	413

The benefits per vehicle-mile of travel saved can be computed by taking into consideration cost of gasoline, oil, tires, maintenance, and that portion of depreciation which is chargeable to driving as distinguished from fixed expenses which must be incurred regardless of the distance traveled. For passenger cars the saving was computed by the Oregon State Highway Commission as 3.21 cents and by the Connecticut State Highway Department as 2.83 cents per vehicle-mile. In this analysis 3 cents is used. The saving to trucks depends upon the type and weight of truck. Figures developed by Oregon are 6.30 cents for light trucks, 8.84 cents for medium trucks, and 14.61 cents for heavy trucks. Connecticut figures are 6.30 cents for light and medium trucks and 13.87 cents for heavy trucks and combinations. In the present analysis the value of distance saved to trucks of all classes combined was assumed to be 8 cents. The daily benefits to automobile users for saving distance would then be as follows:

Passenger cars	$2,012 \times 0.03 = \$60.36$
Trucks	$413 \times 0.08 = \$33.04$
Total	\$93.40

The value of time saved is much more controversial and cannot be computed with a high degree of certainty. However, benefits from time saving are real and are sometimes actually paid for in cash, as in the case of a toll facility. The State highway departments of both Oregon and Connecticut have used a figure of 60 cents per hour as the value of saving time for passenger cars. This figure has had quite general usage and appears to be reasonable on the basis of the meager information available. It is used, therefore, in the analysis of the Martinsville southwest bypass.

The saving for trucks is more definite than that for passenger cars, because there is frequently a money return and the driver is paid. Figures developed by Oregon are \$0.86, \$1.17, and \$1.47 per hour for light, medium, and heavy trucks, respectively, and figures developed by Connecticut are \$0.87 for light and medium trucks combined, and \$1.46 for heavy trucks and combinations. In this analysis \$1.20 per hour, or 2 cents per minute, is used for trucks without classification by size.

Assuming an average speed of 45 miles per hour on the bypass, time saved would be approximately 8 minutes and 45 seconds for all through vehicles which use it. The average daily vehicle-minutes saved during the life of the improvement would be as follows:

Passenger cars	8,525
Trucks	1,775

The total daily benefits from the saving would, therefore, be as follows:

Passenger cars	$8,525 \times .01 = \$85.25$
Trucks	$1,775 \times .02 = \$35.50$
Total	\$120.75

There are numerous other benefits that are not evaluated. Among them are the savings of wear on brakes and tires in eliminating stops, relief from the strain of driving in congested traffic, and benefits to other traffic, particularly that on Church Street, brought about by the lessening of traffic volume by almost 20 percent.

Considering only the distance saving and time saving, the values are as follows:

Distance saving per day	\$93.40
Time saving per day	120.78
Total	214.18
Annual saving	\$78,176

Thus, an annual cost of \$78,176 in the construction and maintenance of a new facility would be offset by the benefit to the users. The relation of first construction cost to annual cost varies in accordance with local conditions and the type of improvement, but, as a rough general figure for improvements of this kind, the annual cost can be considered to be approximately 7 percent of the construction cost, being made up of 3 percent amortization, 2 percent maintenance, and 2 percent average interest figured on the basis of 4 percent each year on the unpaid balance. On this basis an initial construction cost of about \$1,120,000 would be justified by annual benefits of \$78,176.

It may be that a particular improvement is fully justified on the basis of such a calculation, yet should be deferred because other improvements show a higher ratio of benefits to cost. In this case, construction cost estimates have not been made, but, assuming that the route could be constructed for \$100,000, the ratio of benefits to cost would be 11.2. Such a figure derived from an actual estimate could be used as a priority index to be compared with similar indices for other projects.

In the case of Martinsville, the situation is especially favorable for a bypass because of the large saving in distance. Considered on its own merits, such a bypass is well worth constructing and, as has been shown, the benefits would probably be much in excess of the cost. However, it does not solve the Martinsville traffic problem as a whole, nor, in fact, the major part of it. Church Street would be relieved of only 20 percent of its traffic and, while this would afford relief, it would not be many years before increase in traffic volume would make Church Street as congested as it was before the construction of the bypass. Other and more radical measures will have to be taken before the problem is permanently solved.

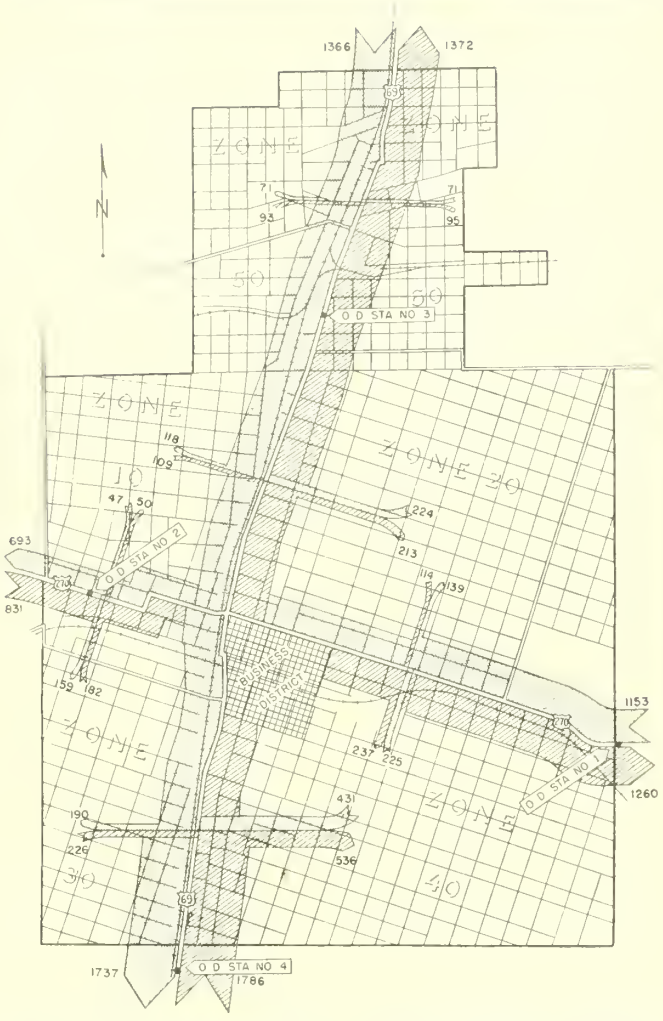


FIGURE 6.—VOLUME OF TRAFFIC ON STATE HIGHWAYS WITHIN MCALESTER, OKLA., HAVING ORIGIN OR DESTINATION OR BOTH OUTSIDE THE CITY.

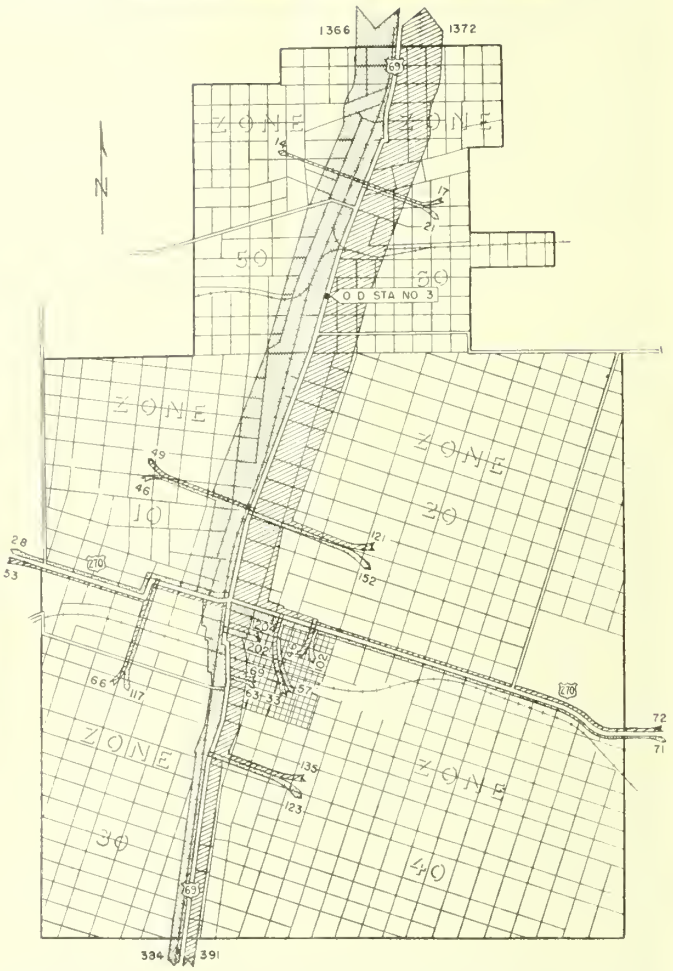


FIGURE 7.—DISTRIBUTION WITHIN AND THROUGH MCALESTER, OKLA., OF IN-BOUND AND OUT-BOUND TRAFFIC PASSING THE NORTH CITY LIMIT ON US 69.

MCALESTER STUDY SHOWS IMPORTANCE OF LOCAL ORIGINS AND DESTINATIONS

A study made in McAlester, Okla., illustrates a more typical situation. McAlester is a town of about 12,000 population, just about the size of Martinsville. A survey very similar to the one made in Martinsville was made there. Interview stations were operated at four points where the State highways enter the city, and questions were asked of motorists concerning origins and destinations.

Figure 6 shows the amount of traffic, with either origin or destination outside the city, which uses different portions of the highways within the city limits. Examination of this chart alone, indicates that a circumferential route would serve a useful purpose and would carry considerable volumes of traffic. However, this is an incorrect conclusion.

Figure 7 shows the distribution of traffic entering and leaving over US 69 at the northern limits of the city. As can be seen, most of this traffic has origin or destination near the center of the city and only a small portion of it goes on through.

Similarly, figure 8 shows the distribution of the traffic entering and leaving the city at its southern boundary on the same highway. Most of this traffic also terminates near the center of the city and the comparatively uniform band which was shown in figure 6 was made up of these two tapering traffic streams superimposed upon each other. A similar situation exists on US 270 running east and west the through traffic being an even smaller portion of the total.

A further analysis of the traffic on US 69 within McAlester is given in figure 9. The narrow ribbon at the bottom shows the through traffic, the jog near the center of the chart being caused by traffic turning off to US 270. The wedge just above this ribbon represents traffic coming in from the north and feeding out into the city, mostly in the central business district. The opposing wedge just above this one represents traffic coming in from the south and feeding into the city in a similar manner. The mound at the top of the chart is the local traffic with origin and destination entirely in the city.

It is evident that, even in a city as small as McAlester, the local traffic constitutes a large portion of the total traffic on the main through routes. For cities of a larger size the local traffic is relatively of much greater importance and the through traffic of correspondingly less importance. Figure 10 shows the traffic on US 66 through St. Louis, Mo. The traffic at the city limits is small compared to that nearer the center of the city, and the large mound represents internal trips almost entirely. Of course, many of the vehicles observed at the city limits are moving locally from a point near the city to a point within the city.

It is clear that the traffic problems of the Federal, State, and local governments merge and cannot be

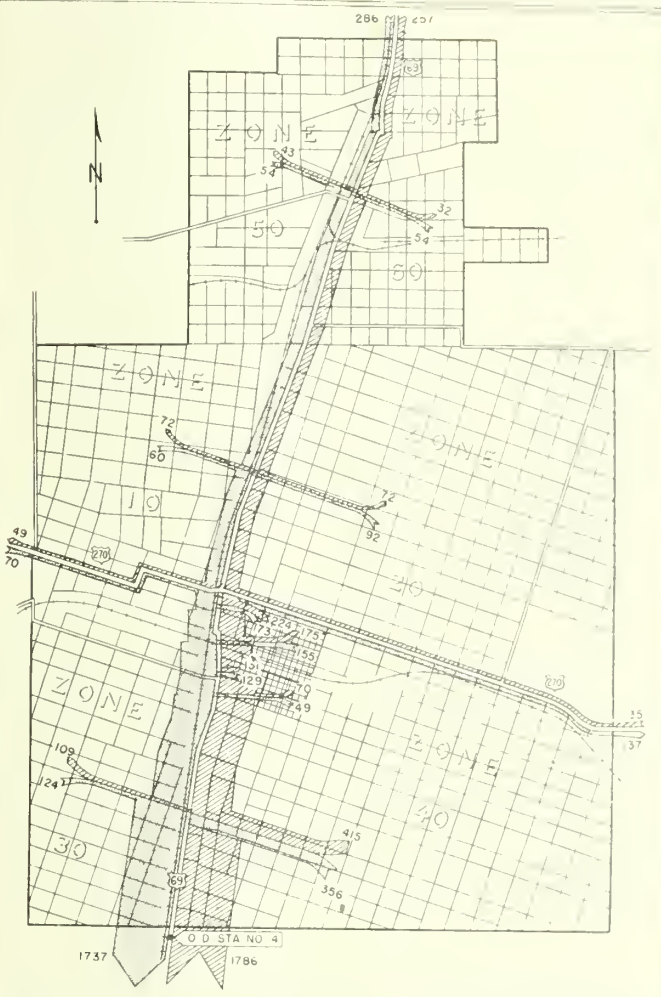


FIGURE 8.—DISTRIBUTION WITHIN AND THROUGH MCALESTER, OKLA., OF IN-BOUND AND OUT-BOUND TRAFFIC PASSING THE SOUTH CITY LIMIT ON U S 69.

separated from each other on any rational basis. When vehicles come into a city from other parts of the State or from other States, they merge with the local traffic, and any facilities built to accommodate them will produce a greater total benefit to the traffic which circulates within the area itself. It is for this reason that State and Federal engineers are having to interest themselves in urban traffic problems which appear to be local in character.

URBAN TRAFFIC COMPLEX

Traffic within an urban area is much more complex than that on rural roads. Traffic volumes are larger, and traffic arteries are much more numerous. Parallel streets offer many alternate routes of travel, and it is not possible to tell from observing traffic volumes alone where the drivers really want to go. Drivers often travel considerable distances out of their way to use exceptionally attractive routes, or to avoid congested and unattractive routes. Examples of this have been noted in numerous cities and origin and destination surveys have shown that the facts were sometimes quite different from assumptions made by traffic experts with long familiarity with the local situation.

Because of the complexity of urban traffic, it is difficult or impossible to collect needed data by methods used at external origin and destination stations. The multiple lanes and the large numbers of vehicles make it difficult and sometimes hazardous to stop traffic and conduct interviews at the roadside. The numerous routes of travel over the grid of streets necessitate operation of a large number of interview stations if all or a large part of the traffic is to be intercepted. Parallel streets invite bypassing survey stations and where roadside interviewing has been attempted at points that could be readily bypassed, there has invariably been bypassing as soon as lines of standing vehicles began to accumulate.

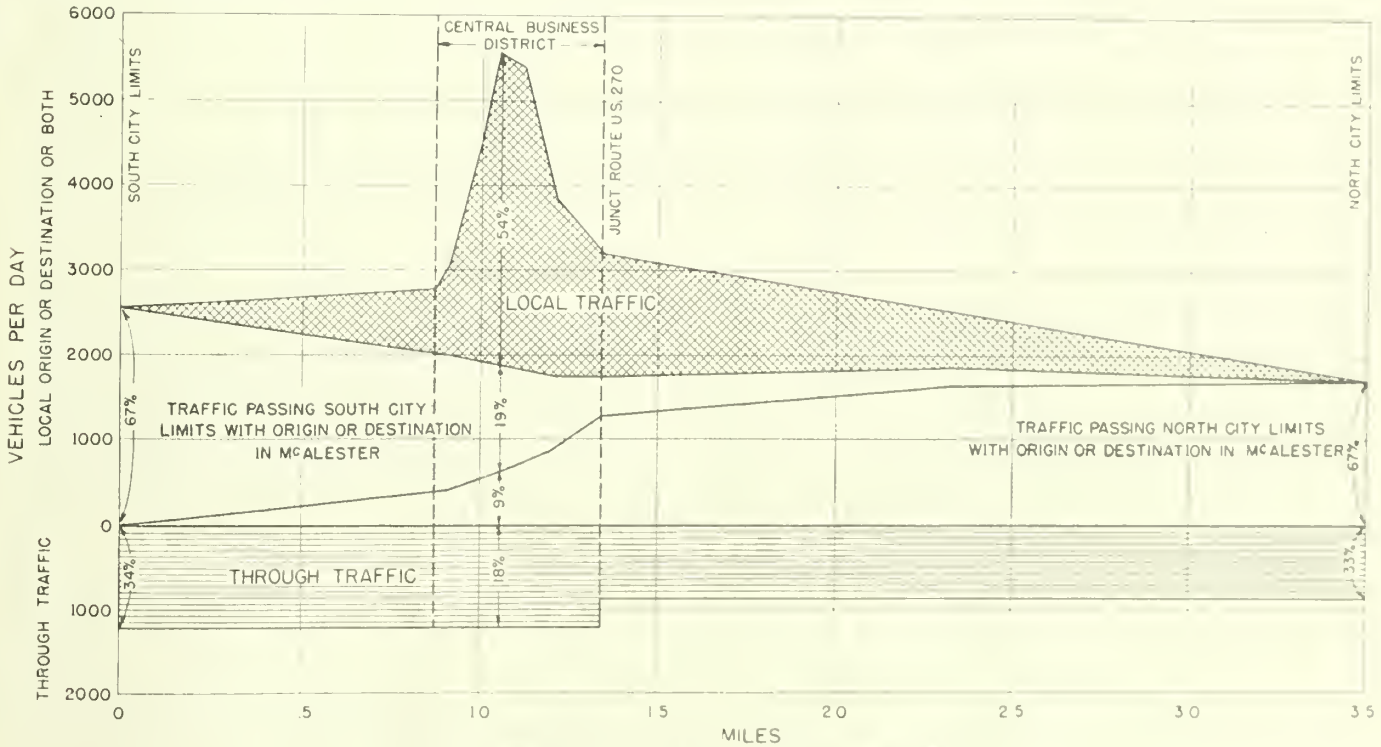


FIGURE 9.—TRAFFIC ON U S 69 IN MCALESTER, OKLA.

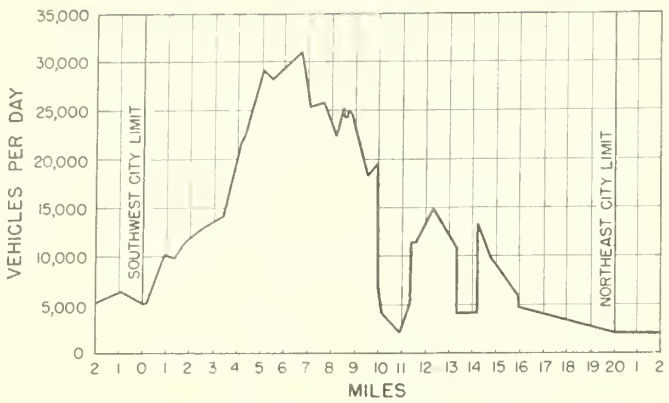


FIGURE 10.—TRAFFIC PROFILE ALONG US 66 THROUGH ST. LOUIS, MO.

For the solution of special problems under especially favorable conditions, it is sometimes possible to collect needed data at traffic-interview stations within a city. In Columbia, S. C., there are a number of points of traffic concentration just outside of the downtown business section, including a bridge across a river. It was possible to operate a cordon of stations around this section in much the same manner as external stations around a city are operated. Some difficulties were encountered, but the work was satisfactorily done even though the traffic volume was as high as 15,000 vehicles per day at one of the stations. The results obtained were limited in some respects, but were sufficient for the solution of the problems under consideration.

Similarly, interviews were conducted at a bridge between Shelton and Derby in Connecticut, and information was obtained which has been quite useful in the solution of highway problems in the Shelton-Derby-Ansonia area.

NEW SURVEY METHODS TRIED

For several years the need for new methods of determining origins and destinations of traffic within urban areas has been apparent and effort has been made to develop improved methods in numerous cities. In Chicago, Ill., the movement of vehicles throughout the city on a typical weekday was determined by observing license numbers as the vehicles passed different survey stations. Several thousand Boy Scouts were used to man the stations and several hundred supervisors were required to direct the work. The results of this survey have been extremely useful in pointing out the location and kind of highway improvements most needed and in assigning priorities to these improvements. One difficulty inherent in the method is that the zones used in traffic observation are necessarily large, and it is not always possible to fix the desired routes of travel with an accuracy as great as is needed.

In Detroit, Mich., questionnaires were given to war workers to determine the daily number of workers traveling between their homes and places of work. The questionnaires were distributed in all major industrial plants through the employers, and the answers showed the location of the employees' homes, how they went to work, participation in group riding, and at what time. To these data were added the origins of downtown employees obtained by the street railway department through a questionnaire distributed in cooperation with building managers, and information concerning shoppers obtained through the department stores. From the combined data the daily number of workers and shoppers going into and out of each of 10 industrial areas and the downtown retail district were determined. The movement to and from each of these 11 areas was portrayed on a map by traffic flow bands, or rays. These were studied and combined in such a manner as to indicate the major routes of travel. This chart was superimposed on a map of existing streets and from it a network of proposed expressways was developed.

In Cleveland, Ohio, a traffic study, very similar to that in Detroit, was made. Data from worker questionnaires were used in determining expressway location. This survey was made before the Detroit survey and was completed before the war. As a result of experience in Cleveland, the Ohio Highway Department decided that it would be better to interview persons in their homes rather than to seek information at their places of employment. In Toledo, Ohio, home interviews were obtained by having school children bring in reports on the daily travel to and from work by all the workers in their families.

All of the methods described have been extremely useful and have met the needs for which they were made. None of them, however, are sufficiently comprehensive to serve as a desirable basis for long-range highway planning envisioning the expenditure of large amounts of Federal, State, and local funds. When it became apparent that urban construction would play an important part in the postwar program, a study was made of all methods of urban traffic studies that have been used, and a new method was developed based on this experience.

HOME INTERVIEW METHOD DEVELOPED

The work which had been done indicated that the best line of approach was to interview occupants of dwellings according to a standardized method of sampling, and to determine the travel by automobile or by public conveyance by each resident of the dwelling unit on a representative weekday. Since the Bureau of the Census has had a large amount of experience with home interviews on a sample basis, conferences were held with officials of that bureau. In surveys made by the Bureau of the Census, and participated in by numerous nationally known public opinion experts, it was determined that a sample would be representative if it were uniformly distributed as to area. A consumer requirement survey, for example, showed that in a sample selected on the basis of place of residence alone, the relations between age groups, family size, sex, race, and occupational groups were essentially the same as determined in the full census. It was decided, therefore, to distribute the sample uniformly as regards place of residence and to ignore all other factors, such as occupation. To be representative the sample must be preselected and rigidly followed. The interviewer should have no option as to whom he will interview.

Following the conferences with the Bureau of the Census, conferences were held with State and city highway officials in Little Rock, Ark.; Tulsa, Okla.; New Orleans, La.; and other cities. A manual of procedure was developed and work was started in the cities mentioned in February or March of 1944. Interest has spread rapidly and surveys of this type are now being made, or have recently been completed in

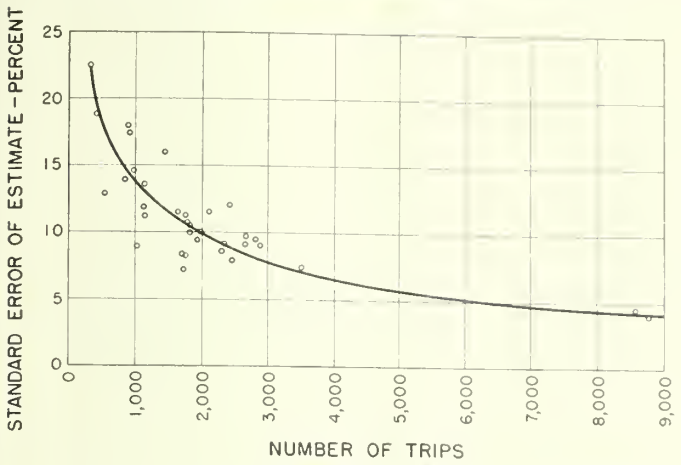


FIGURE 11.—STANDARD ERROR OF ESTIMATE OF TRIPS DESTINED TO DIFFERENT ZONES IN SAVANNAH, GA.

the metropolitan areas of 35 cities.[1] The manual of procedure is being constantly revised on the basis of experiences gained, but the basic methods remain unchanged.

Information concerning traffic entering and leaving an area is determined at a cordon of external stations located on the principal highways at the boundary of the area. The method used is to conduct roadside interviews in the manner described for Martinsville and McAlester.

Information concerning trips made wholly within the area by persons residing there is determined from home interviews conducted in dwellings selected to form a representative sample. The size of the sample as a percentage of the total varies in accordance with the size of the urban area. In cities of 100,000 to 300,000 population a 10-percent sample has generally been used. In the smaller cities the sample has generally been larger, while in the very large cities samples of 5, 4, and 3⅓ percent are used. As data from the surveys become available, analyses are made to aid in determining the size of sample best suited to give the desired balance between accuracy and cost.

The basis for the sample selection is a map showing the streets and all of the residences. If the sample is 1 in 10 in a zone of one-family houses, every tenth house is selected proceeding clockwise around a block. If there are apartment houses, each apartment is treated as though it were a separate dwelling unit and the counting proceeds through the apartment house, selecting every tenth apartment.

Each interviewer is given a list of the dwelling units in his territory selected for inclusion in the sample, and he is instructed to obtain interviews in these dwelling units and no others. He obtains information concerning all trips made by automobile or by public conveyance on the preceding day, including the means of travel, the origin and destination, the purpose of the trip, the times of starting and arrival, and other information which varies somewhat from city to city. There have been refusals to give the information in only a negligible number of cases.

In the home interviews no information is obtained concerning truck travel, but a special sample of trucks

[1] Boston, Providence, Newark, Baltimore, Richmond (Va.), Charlotte (N. C.), Greenville (S. C.), Spartanburg (S. C.), Atlanta, Savannah, Waycross (Ga.), New Orleans, Baton Rouge, Shreveport, Memphis, Nashville, Chattanooga, Cincinnati, Indianapolis, Fort Wayne, South Bend, Port Huron (Mich.), Milwaukee, Ottumwa (Iowa), Mason City (Iowa), Omaha, Lincoln (Nebr.), St. Louis, St. Joseph, Springfield (Mo.), Kansas City, Little Rock, Oklahoma City, Tulsa, and Denver.

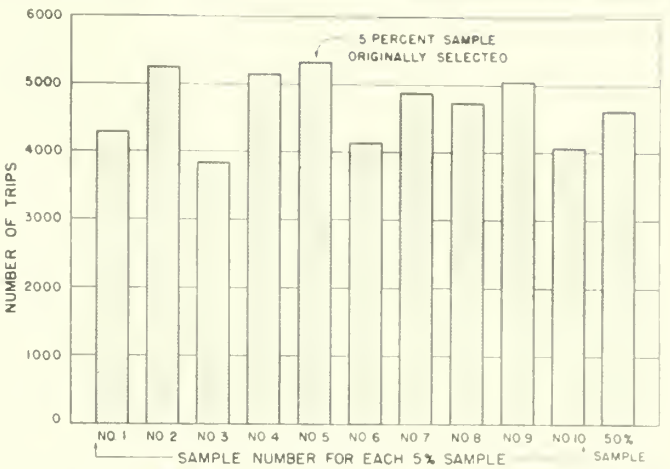


FIGURE 12.—COMPARISON OF THE NUMBER OF TRIPS IN TEN EXPANDED 5-PERCENT SAMPLES IN TRACT 97, MILWAUKEE, WIS.

is selected from the registration list. The owners of the trucks selected are called on by an interviewer and information concerning all of the trips made by each truck on the day preceding the interview is recorded. If there is no record of the trips made by the selected truck on that day, the owner is asked to have the driver keep a record on the day following and this information is used instead. The data concerning truck trips generally consists only of the type of commodity carried, the origin and destination, a list of the stops made, and the time of starting and ending the trip. Taxicabs are sampled and data obtained concerning their movements in the same manner as for trucks.

SURVEY ACCURACY TESTED

In several cities checks have been made to test the adequacy of home-interview samples. The method used was based on recommendations of the Bureau of the Census and involves sampling the sample to derive factors indicating probability of error. Figure 11 shows for Savannah, Ga., the amount which estimates of the number of trips made to different zones, based on a 10-percent sample, would be expected to vary from the values which would have been obtained if interviews had been made in every household. The abscissas of the curve represent the number of trips with destination in a given zone and the ordinates represent the standard error of estimate, which is the percentage variation which would not be exceeded oftener than about one-third of the time. Naturally the larger the number of trips being estimated, the more accurate the estimate can be expected to be. For a group of about 6,000 trips the standard error is only about 5 percent, whereas for a group of about 1,000 trips it is about 14 percent. On the whole, the curve indicates that a sample of 10-percent is adequate for a city the size of Savannah because it is not probable that there will be need to estimate traffic volumes (or trip groups) less than several thousand vehicles per day with a very small percentage of error.

In Milwaukee, Wis., the sample for the city as a whole was 5 percent. Ten 5-percent samples were selected in one of the zones and variations between these different 5-percent samples were determined. Figure 12 shows the number of trips made by residents of the zone as determined from each of the samples. The extreme variation is from about 3,900 (sample 3) to about 5,300 (sample 5). The probability of obtain-

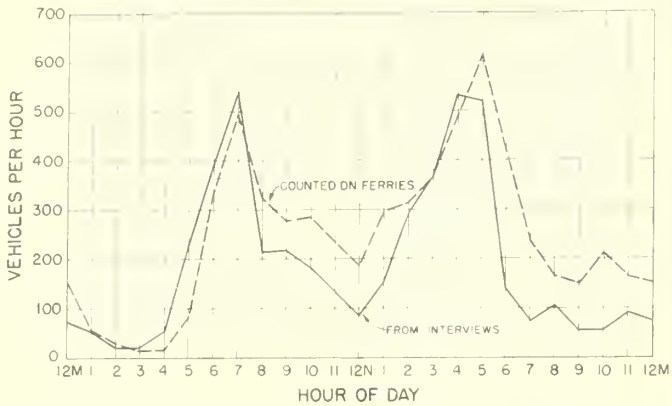


FIGURE 13.—COMPARISON OF THE ACTUAL NUMBER OF PASSENGER CARS CROSSING THE MISSISSIPPI RIVER ON THE FERRIES AT NEW ORLEANS, LA., WITH THE NUMBER ESTIMATED FROM INTERVIEWS TO HAVE DONE SO.

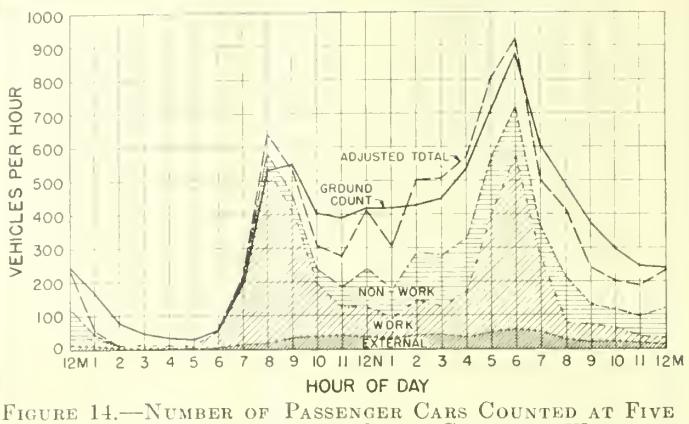


FIGURE 14.—NUMBER OF PASSENGER CARS COUNTED AT FIVE CONTROL POINTS IN DENVER, COLO., COMPARED WITH THE NUMBER ESTIMATED FROM INTERVIEWS TO HAVE PASSED THESE POINTS.

ing a sample that deviates from the mean as much as No. 3 is only about 5 percent, and the probability of obtaining one that deviates from the mean as much as No. 5, at the other extreme, is only 7 percent. The test for Milwaukee seems to indicate that a 5-percent sample is in keeping with the over-all requirements as to cost and accuracy.

In order to test the completeness with which trips are reported, certain well-known points through which traffic funnels are designated as control points and, in the interviews, information is obtained concerning the vehicles passing them. Counts are made at these points and the expanded interview data are compared with the actual ground counts. A bridge or an important artery carrying a large amount of intracity traffic is ideal for a control point. In some cases there are no points of traffic concentration that are well enough known to make them suitable control points. It is then necessary to determine from origins and destinations alone those trips crossing a so-called "screen line" drawn across the area and also to count all of the trips on all of the streets crossing this line, making as many counts as there are streets crossing the line. A river makes an excellent screen line and, in some cases, this method of checking is preferable to the control point method. One objection to it is that it is sometimes possible for a trip to cross the line twice, even though the origin and destination are both on the same side of the line, and the line has to be selected with considerable care to minimize occurrences of this kind.

Figure 13 shows for New Orleans, La., expanded interview data by hours and compares these results with counts on all ferries crossing the Mississippi River, the river being used as a screen line. The morning and afternoon peaks check very closely, but a considerable portion of the midday and night travel appears to have been missed in the interviews. Since much of the travel in the morning and afternoon peaks is to and from work, and much of that in the midday and night periods is for shopping and recreational purposes, this chart suggests that work travel was more fully reported in the interviews than nonwork travel. This was expected in view of the restrictions on driving in effect when the survey was made. As a test of this conclusion a zone was selected in an industrial area containing only manufacturing plants for which employment figures could be obtained. This zone was census tract No. 17 containing the Delta, Higgins, and Pendleton shipbuilding plants and five smaller plants. According to the manufacturers, the total number of daily workers was 26,149, some of whom may have walked to work. According to actual counts made by the New Orleans Public Service Commission, the number of people riding to work daily by automobile, streetcar, or bus was 23,531. The expanded home-interview data indicated work trips into this zone amounted to 21,904 per day. The percentage relation between those driving by automobile and those riding by streetcar or bus was almost the same in the Public Service Commission data as it was in the survey data. This confirms the implication of figure 13 that trips to work were almost completely reported and that the trips not reported were those for recreation and other nonwork purposes.

Figure 14 shows a comparison between ground counts of passenger cars and expanded interview data for five control points combined in Denver, Colo. Here again the ground counts and expanded interview data are in agreement for the morning peak and the difference between them is not great in the afternoon peak. Apparently, trips were missed in the interviews for the midday and night periods, and presumably, these were trips other than to and from work. If the figures for nonwork travel were adjusted by a factor of 2.5, the interview data would be in fairly close agreement with the ground counts as indicated by the line on the chart labeled "adjusted total."

Fortunately, from the point of view of design requirements, traffic during peak hours is being determined rather closely from the expanded interview data and the trips missed are generally those occurring in off-peak hours. However, an effort is being made to improve techniques to obtain more complete information concerning trips throughout the day. That such efforts are bearing fruit is indicated by figure 15 showing combined data for three control stations in Milwaukee, where the survey was made later. This check indicates that the data are sufficiently accurate to be used without adjustment.

Another indication that more complete information is being obtained is that the number of trips per dwelling unit averages much higher in the later than in the earlier surveys. In Milwaukee and Richmond, for instance, about five trips per dwelling unit were obtained compared to about three trips per dwelling unit in New Orleans and Denver. Improvements are being brought about by more careful training of the interviewers, insistence that the information be obtained

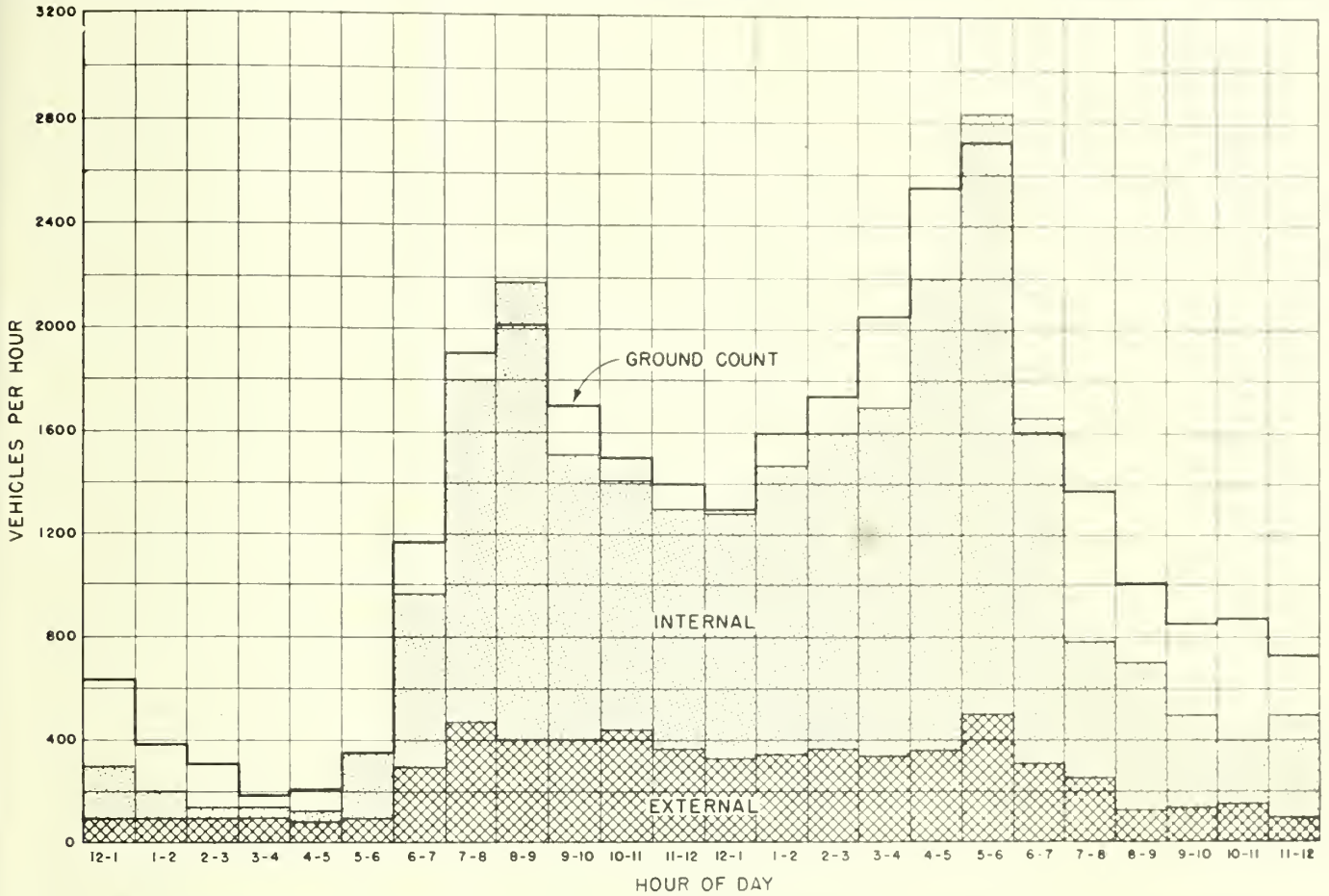


FIGURE 15. COMPARISON OF THE NUMBER OF PASSENGER CARS AND TRUCKS COUNTED AT THREE CONTROL POINTS IN MILWAUKEE, WIS., WITH THE NUMBER ESTIMATED FROM INTERVIEWS TO HAVE PASSED THESE POINTS.

first hand from the person who did the traveling, even at the expense of more repeat calls, and, in some cases, by a shortening and simplification of the form.

It was necessary to undertake the surveys under war conditions in order to develop information that would be of value in the immediate postwar program. Because of manpower shortages, particularly in supervisory capacities, it was necessary to limit the work to procurement of information concerning the most important problems. In all cases, field work was conducted for only a few months and there is a lack of seasonal coverage. Because of war restrictions, Sunday and holiday travel was curtailed and data were obtained for weekdays only. It will be necessary to make allowances for these conditions in interpreting the data. As regards the shift from automobile to public transportation because of gasoline rationing, the basis for adjustment is contained in the reports of interviews. Full information was obtained concerning trips by all means and the desired method of travel under normal conditions.

Now that the war is over it will undoubtedly be practicable to use a well-trained party and to extend the work over a full year, obtaining information concerning Sunday and holiday travel, as well as weekday travel. If this can be done, a degree of accuracy much greater than any yet obtained should be possible.

USES OF THE DATA

The most useful tabulations made from survey data show the number of trips daily from one zone to another by automobiles, trucks and passengers in automobiles and public transit vehicles. From these tabulations determination can be made of routes that will best serve traffic needs, and the traffic volumes to be expected on new facilities which may be constructed. Such data for the greater Kansas City area are depicted graphically in figures 16 to 20, inclusive.

Figure 16 shows the interzone movements of passenger cars occurring wholly within the area which amount to 300 or more trips per day; figure 17 shows those which consist of 150 to 299 trips per day, and figure 18 shows those of 75 to 149 trips per day. The major movements depicted in figure 16 follow a well-defined pattern, while the minor movements are shown by figure 18 to be widely dispersed. It is obvious that movements of less than 75 trips per day are so widely scattered that no pattern or trend can be shown. No matter where improvements are located the minor traffic movements will be served in about equal proportions, and it is the larger movements shown in figures 16 and 17 that should govern in route selection.

Figures 19 and 20 give similar information for passenger-car trips passing external stations and extending beyond the limits of the area. From the data depicted in figures 16 to 20, inclusive, figure 21 was constructed to show major desire lines of travel for passenger automobiles. If it were possible to establish adequate routes along these lines, they would attract approximately the volumes of passenger-car travel indicated by the band widths. This and a chart showing the desired lines of travel for trucks, similarly constructed, form the basis

FIGURE 16.—DESIRED LINES OF AUTOMOBILE TRAVEL BETWEEN ZONES IN KANSAS CITY HAVING A VOLUME OF 300 OR MORE TRIPS PER DAY.

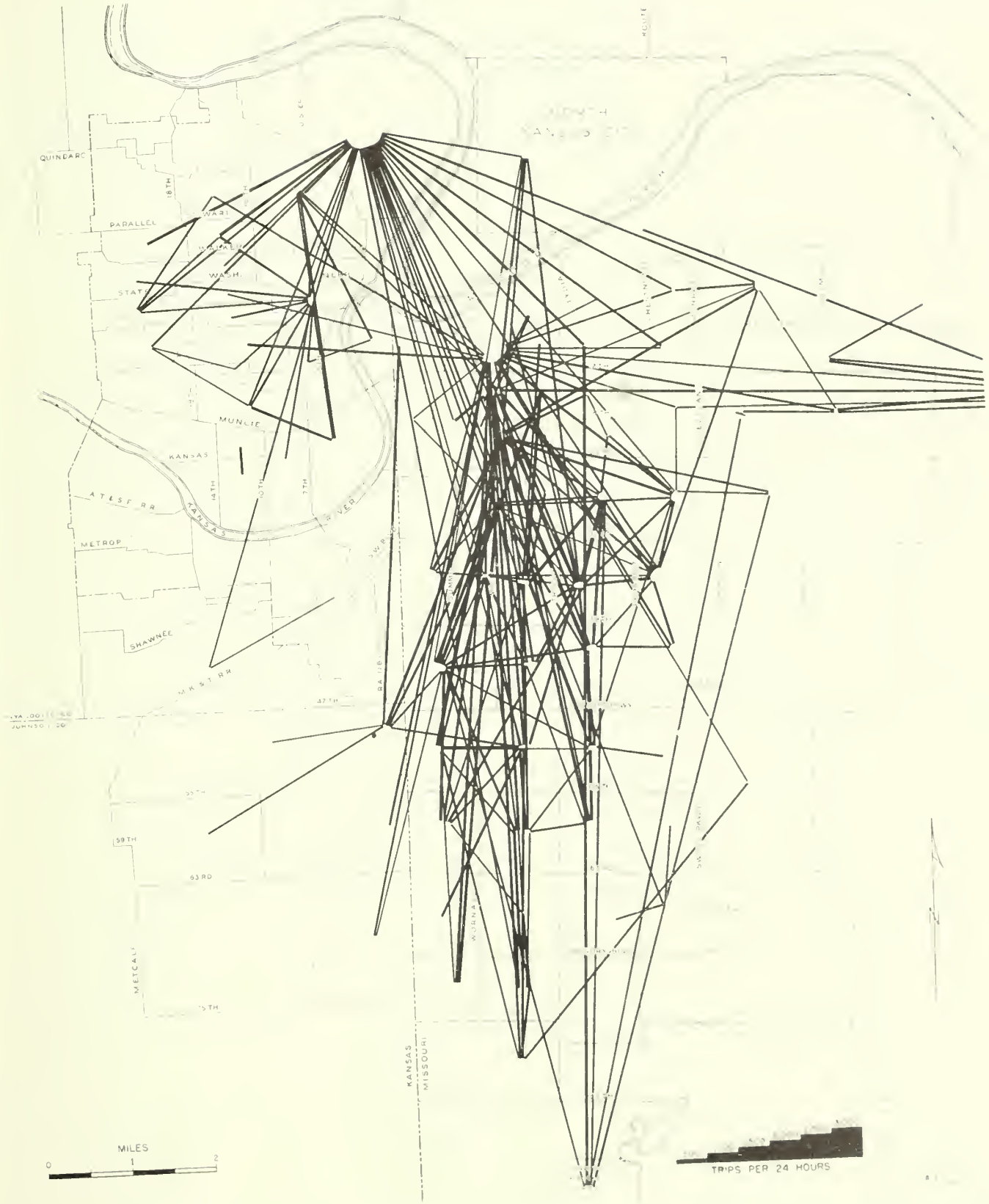


FIGURE 17.—DESIRED LINES OF AUTOMOBILE TRAVEL BETWEEN ZONES IN KANSAS CITY HAVING A VOLUME OF 150 TO 299 TRIPS PER DAY.

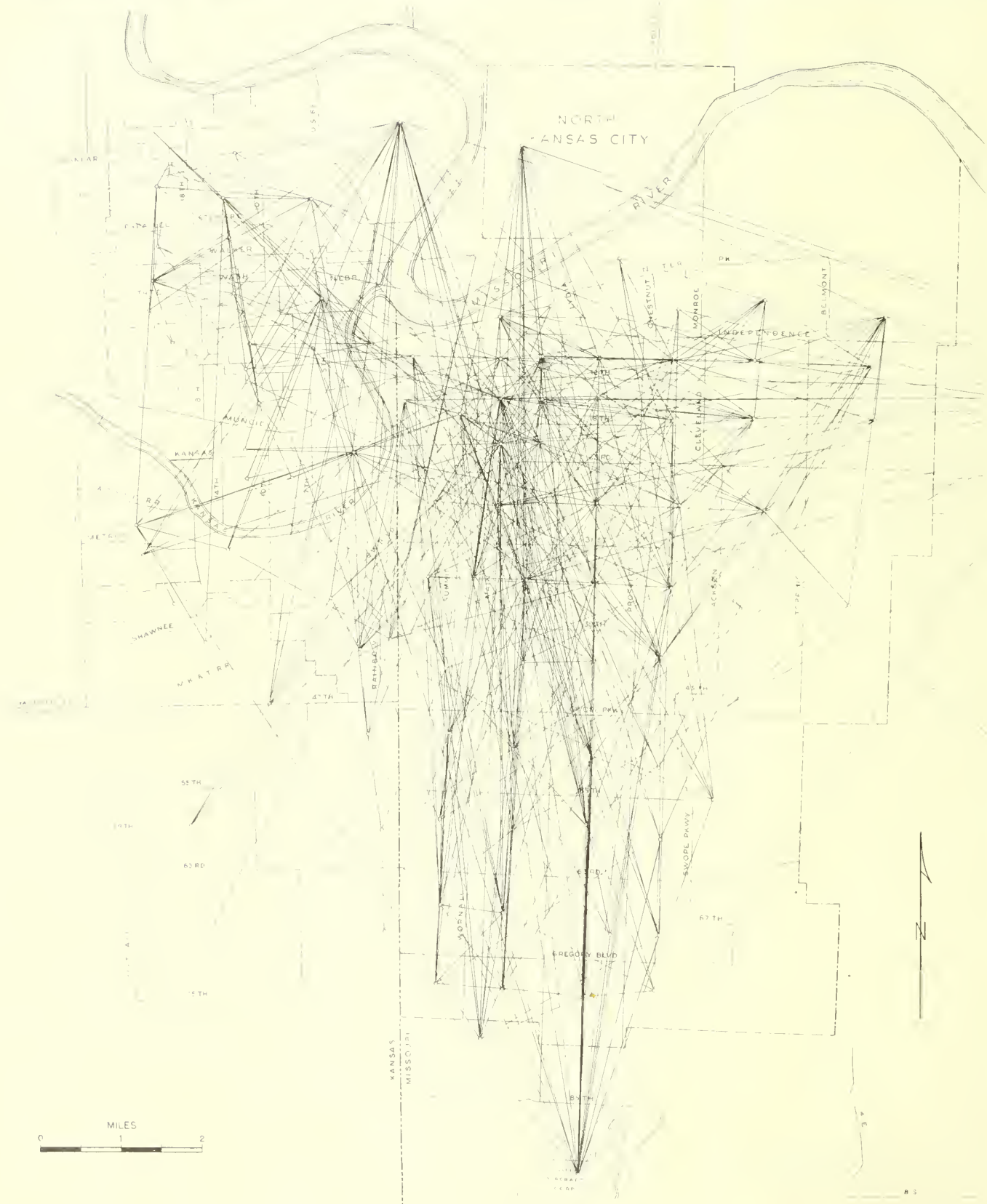


FIGURE 18.— DESIRED LINES OF AUTOMOBILE TRAVEL BETWEEN ZONES IN KANSAS CITY HAVING A VOLUME OF 75 TO 149 TRIPS PER DAY.

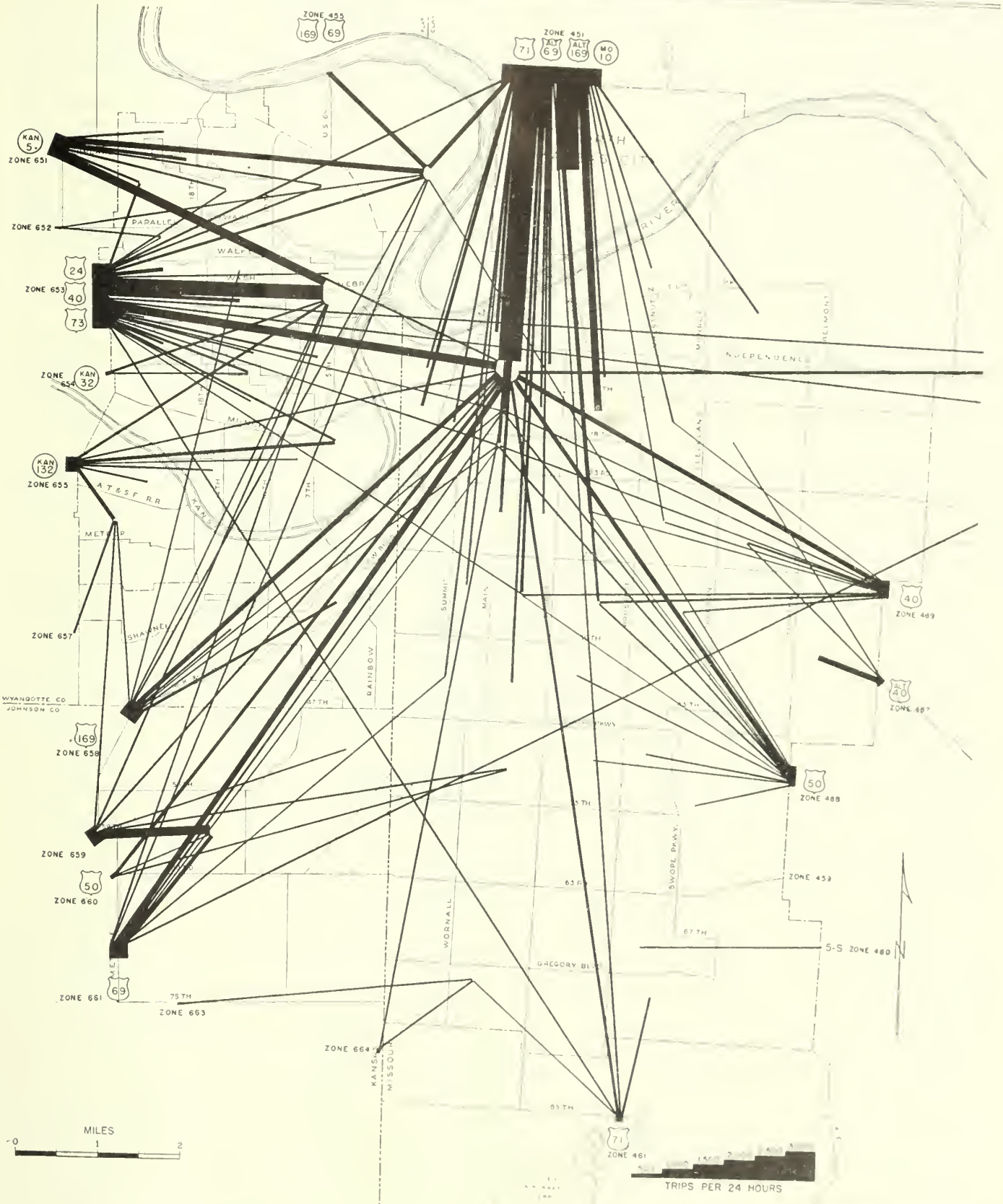


FIGURE 19.— DESIRED LINES OF AUTOMOBILE TRAVEL FROM ZONES IN KANSAS CITY TO POINTS OF EXIT FROM THE CITY HAVING A VOLUME OF 100 OR MORE TRIPS PER DAY.

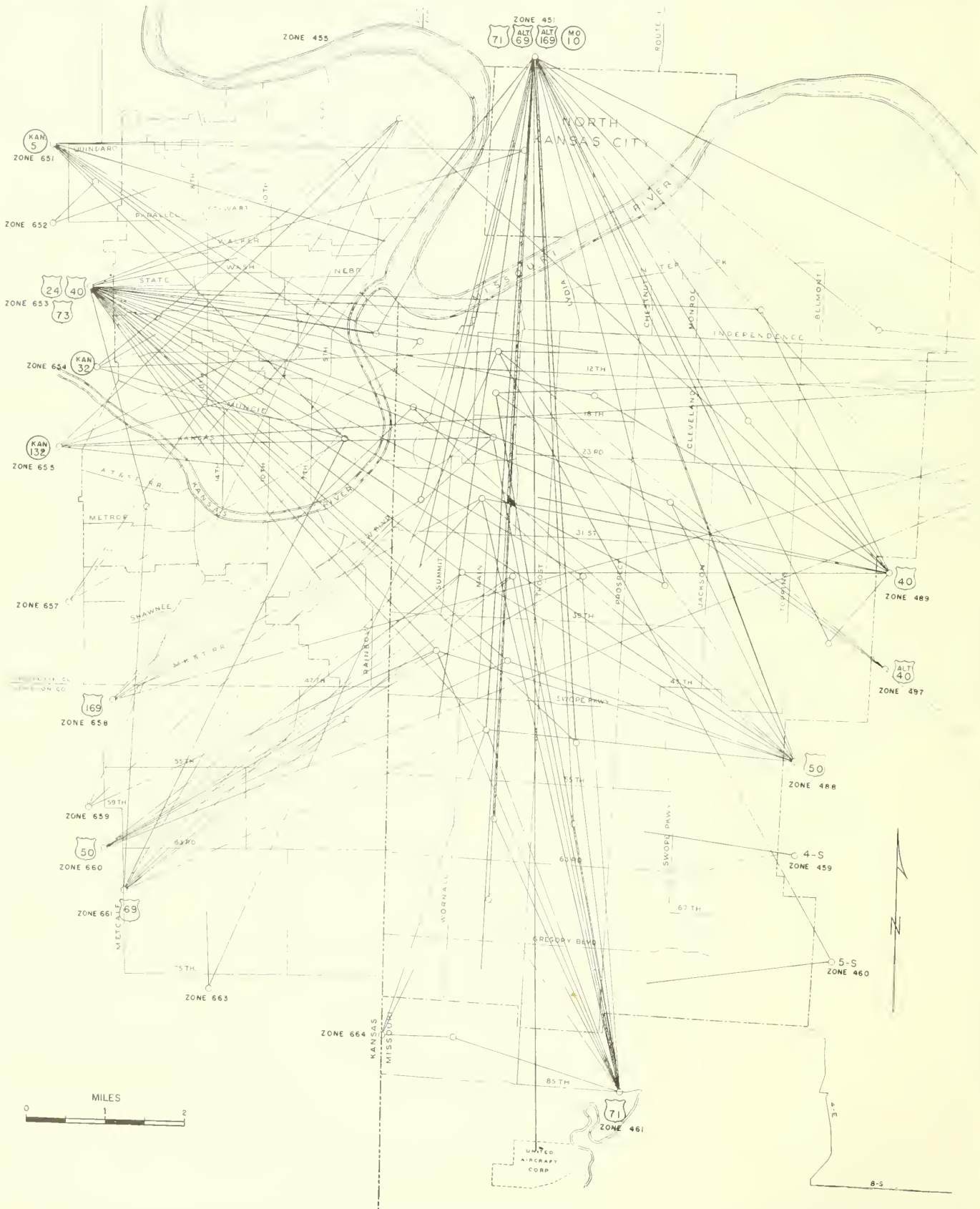


FIGURE 20.—DESIRED LINES OF AUTOMOBILE TRAVEL FROM ZONES IN KANSAS CITY TO POINTS OF EXIT FROM THE CITY HAVING A VOLUME OF 50 TO 99 TRIPS PER DAY.

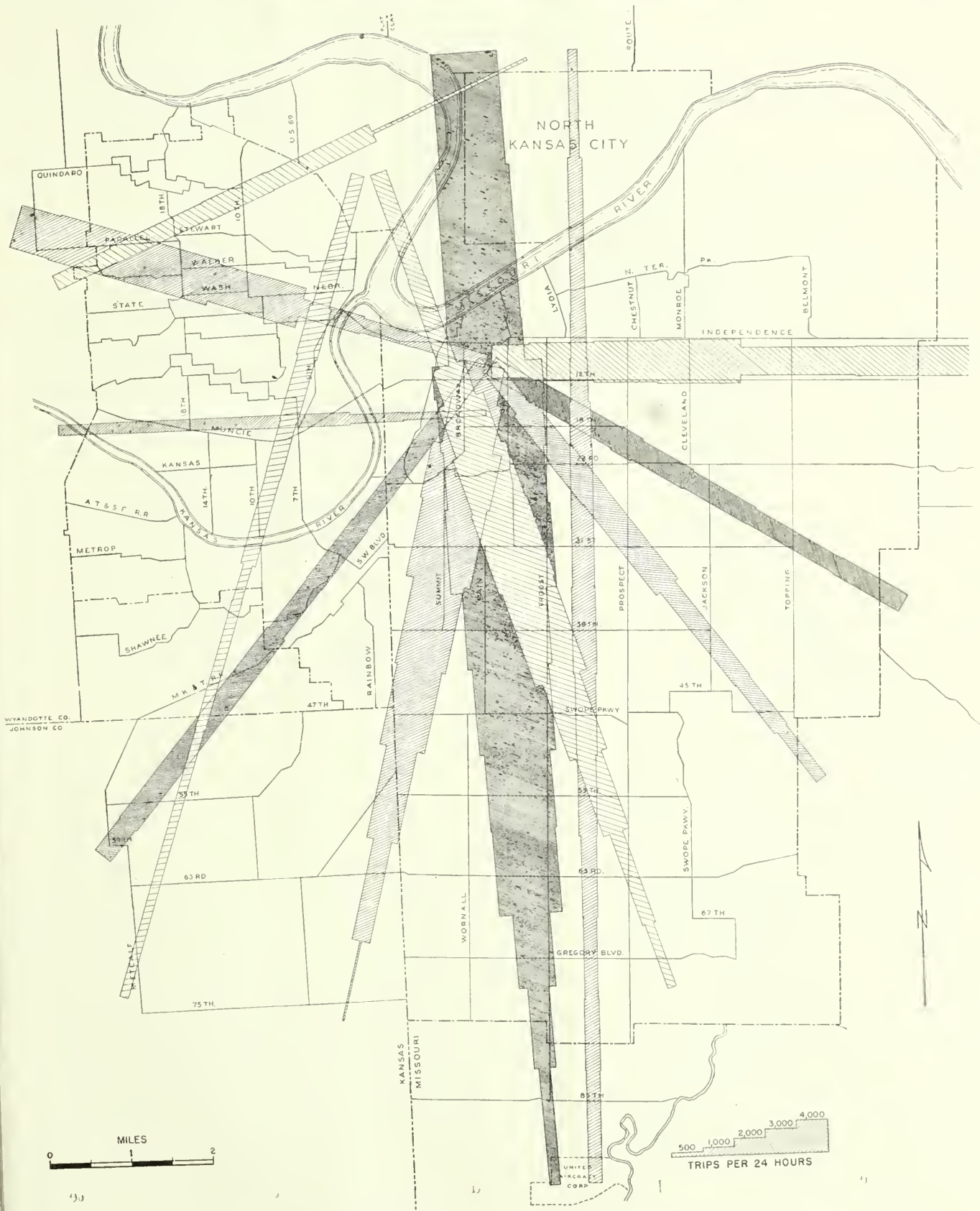


FIGURE 21.—MAJOR DESIRED LINES OF AUTOMOBILE TRAVEL IN KANSAS CITY.

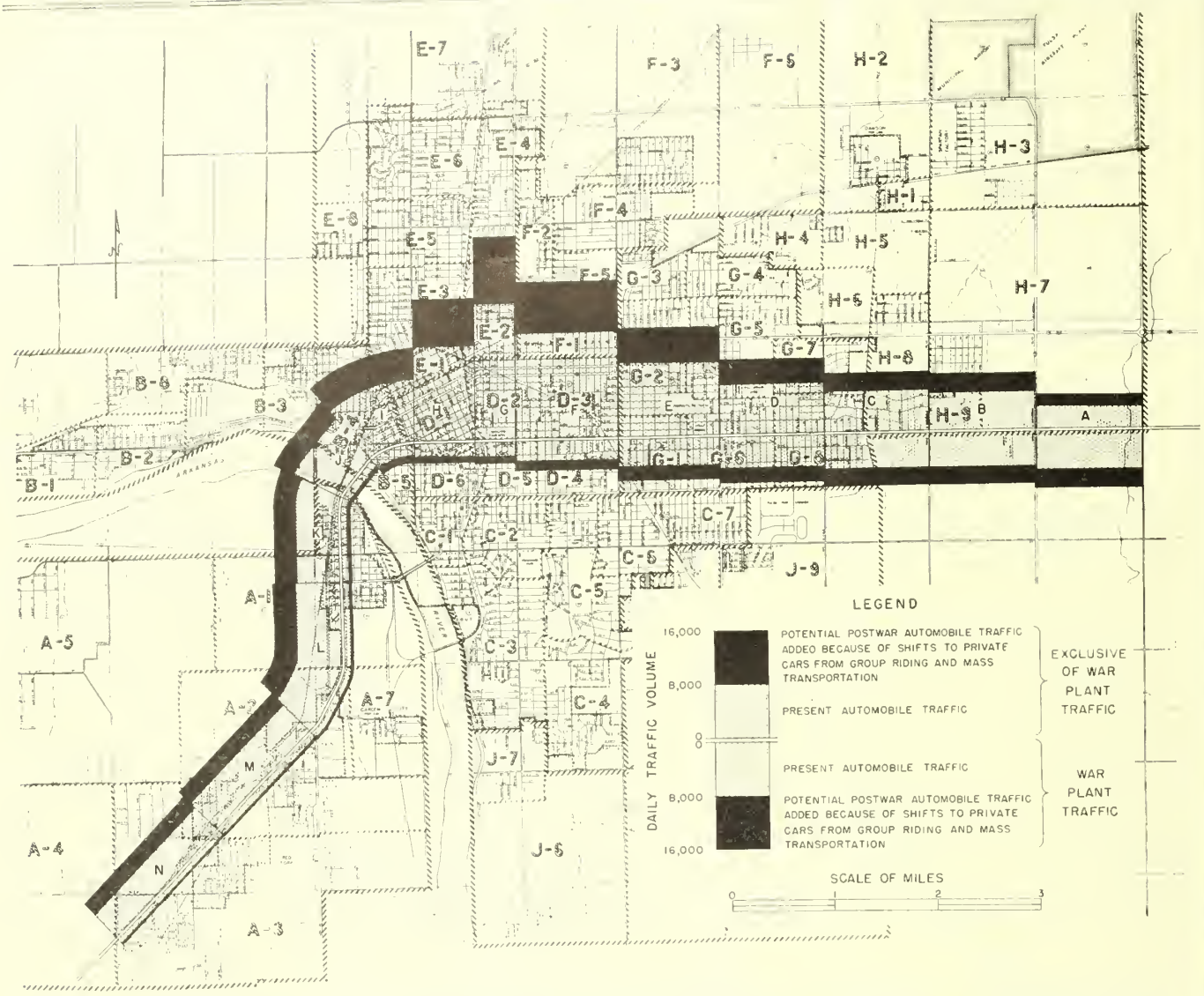


FIGURE 22.—ESTIMATED VOLUME OF EXISTING AUTOMOBILE TRAFFIC THAT WOULD USE AN EXPRESSWAY LOCATED APPROXIMATELY AS SHOWN IN TULSA, OKLA., AND THE AMOUNT OF INCREASE THAT MAY BE EXPECTED IN THE IMMEDIATE FUTURE.

for the location of expressways from the viewpoint of traffic service. The final selection of routes will be influenced by considerations of topography and right-of-way costs, as well as traffic criteria.

An analysis was made to determine the traffic which would use an expressway in Tulsa, Okla., if constructed approximately along the route of U S 66. Figure 22 shows the result as determined from the zone to zone traffic flow. In estimating this traffic it was assumed that the expressway would be used whenever time would be saved by doing so, even though the distance were somewhat longer. There was a large war plant in the northeastern part of the city, and, as traffic to this plant might be temporary, it was thought advisable to treat it separately from other traffic. This war-plant traffic is shown below the white line indicating the proposed route of the expressway, and all other traffic is shown above this line. Traffic to the war plant is comparatively small in the downtown section (zone D–1) and becomes relatively large at the edge of the city, whereas the other traffic is relatively small at the edge of the city and increases as the downtown section is approached. The lighter portion of the band adjacent to the white line represents existing automobile travel that would use the route, and the black part at the outer edges represents trips made by passengers in automobiles, streetcars, and busses which would probably become automobile trips with the removal of wartime restrictions. This potential traffic was estimated on the basis of answers to a question asked in the interviews and a comparison of the relative volumes of travel by automobile and by public conveyance in the prewar period and at the time of the survey.

In addition to the expressway location shown on this chart, other locations were analyzed in a similar manner in order to determine the service which would be rendered by each. The traffic shown in figure 22 is that which might be expected to develop soon, now that gasoline rationing is over and new cars are in prospect. Estimates are yet to be made of probable traffic during the life of the improvement. As the design progresses, the traffic that will use interchanges at various locations can be estimated. When properly analyzed and interpreted, the survey data give all of the information needed to compare a route with any other route as regards traffic service, to determine the design volumes for different sections, and to determine the needed locations and capacities of the interchanges.

U. S. GOVERNMENT PRINTING OFFICE: 1945

JOINTS AND CRACKS IN CONCRETE PAVEMENTS

A COOPERATIVE STUDY BY PUBLIC ROADS ADMINISTRATION AND THE MICHIGAN STATE HIGHWAY DEPARTMENT

Reported by EARL C. SUTHERLAND, Senior Highway Engineer

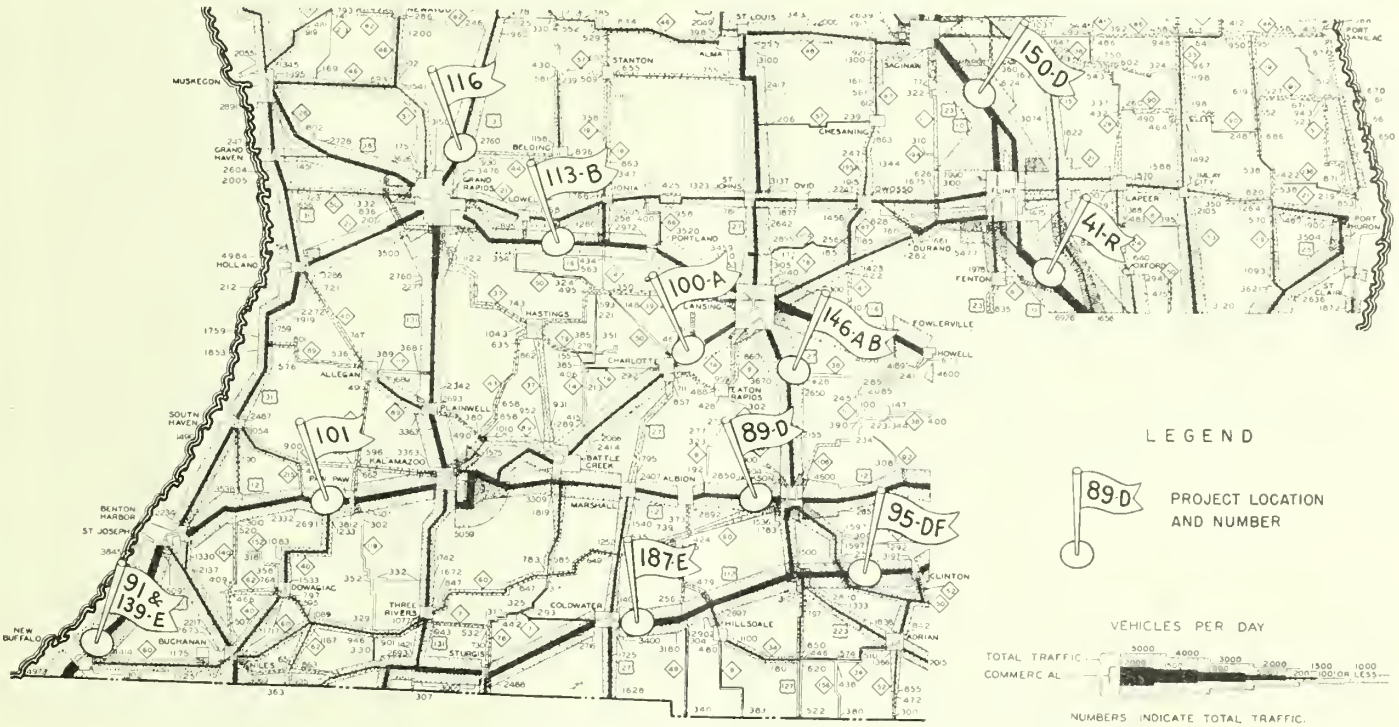


FIGURE 1.—LOCATION OF PROJECTS STUDIED AND AVERAGE TRAFFIC VOLUME.

DURING THE SUMMER OF 1938 a cooperative study was undertaken by the Michigan State Highway Department and Public Roads Administration, primarily to develop information concerning the condition of transverse joints in concrete pavements in which the joints had been constructed without provision for load transfer. Michigan has in its highway system a number of such pavements that have been in service for a long period of time, some of which carry relatively heavy traffic.

The selection of projects to be included in this study was made after a preliminary survey of the concrete pavements in the southern part of the State.[1] The majority of the projects included are approximately 10 years old and all of them have been subjected to comparatively heavy traffic during the entire period. The numbers and locations of the projects studied are shown on the accompanying traffic map, figures 1 and 2, which also indicate the average 24-hour daily commercial traffic flow. The traffic survey data from which this map was made were collected between January 20, 1936, and January 15, 1937.

General information concerning the projects and certain features of design of the pavement slabs are given in table 1. Special features of design of some of the projects are shown in table 2. The two tables should be considered jointly.

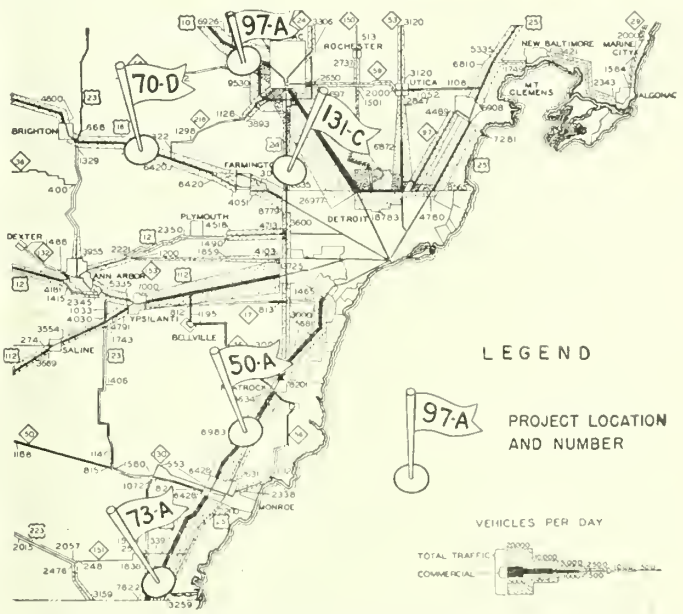


FIGURE 2.—LOCATION OF PROJECTS STUDIED IN VICINITY OF DETROIT AND AVERAGE TRAFFIC VOLUME.

[1] The preliminary survey was made by H. C. Coons of the Michigan State Highway Department, together with L. P. Scott, A. F. Haelig, L. W. Teller, and E. C. Sutherland of Public Roads Administration. The subgrade survey was made by A. E. Matthews, Assistant Engineer of Soils, Michigan State Highway Department. The crack survey was made by G. A. Mansfield, Assistant Research Engineer, Michigan State Highway Department. The joint measurements and general observations were made by A. F. Haelig, Senior Highway Engineer, and E. C. Sutherland, Senior Highway Engineer of the Public Roads Administration.

680471—46——1

TABLE 1.—*Description of the projects*

Federal-aid project number	County	US route number	Year built	Length	Width	Cross section [1]	Sections studied, station to station	Reinforcement and edge bars	Expansion joints design data	Load transfer	Contraction joints	Longitudinal joints
				Miles	*Feet*	*Inches*						
100-A	Eaton	27	1925	5.3	20	9-7-9	82+00—134+92 162+09—217+89 269+96—324+33	[2]	1 inch at 100 feet	Station 111+20—340+100, ¾-inch bar at each edge. None 66+00—111+20.	None	Deformed metal plate. Tie bars, 5-foot spacing.
113-B	Ionia	16	1926	12.0	20	9-7-9	1402+42—1455+70 1613+87—1666+83 1881+12—1934+44	None	do	None	do	Do.
116	Kent	131	1924	4.4	20	7	14+30—110+23 170+38—223+43	¾-inch edge bars, continuous	¾ inch at 50 feet	Edge bars continuous	do	Do.
101	Van Buren	12	1924	8.4	20	8 9-7-9 8	90+50—143+43 226+67—279+44 336+83—390+54	[2]	¾ inch variable	Edge bars continuous in certain sections.	do	Do.
139-E	Berrien	12	1931	2.5	40	9	70+50—176+00	Reinforced throughout with 60-pound wire fabric.	1 inch at 100 feet	None	do	[2]
91	do	12	1923	1.3	18	8	0+00—53+66	¾-inch edge bars	None		do	Deformed metal plate. Tie bars, 5-foot spacing.
91 [3]	do	12	1931	1.3	22	9	0+00—53+66	Reinforced throughout with 60-pound wire fabric.	1 inch at 100 feet	None	do	[2]
89-D	Jackson	12	1924	5.7	20	8	1260+37—1313+30 1371+71—1425+05 1475+99—1529+44	None	¾ inch at 100 feet	do	do	Deformed metal plate. Tie bars, 5-foot spacing.
95-D F	Lenawee	112	1926	13.5	20	9-7-9	1140+52—1193+51 1275+04—1328+13 1500+93—1553+90	do	1 inch at 100 feet	do	do	Do.
73-A	Monroe	24	1925	5.5	20	10-8-10	64+79—141+54 198+25—278+26	do	do	do	do	Deformed metal plate. Tie bars, 30-inch spacing.
73-A [3]	do	24	1935	5.0	11	10-8-10	64+79—141+54 198+25—278+26	Reinforced throughout with 60-pound wire fabric.	1 inch at 50 feet	Translode	50-foot spacing.	None. [4]
50-A	do	24	1923	8.7	20	9	238+64—291+78 400+67—453+53 606+15—658+80	¾-inch edge bars and special [2]	Variable spacing. Width not known.	None	None	Deformed metal plate. No tie bars.
50-A [3]	do	24	1933	8.4	20	10	238+64—291+78 400+67—453+53 606+15—658+80	Reinforced throughout with 60-pound wire fabric.	1 inch at 100 feet	do	do	None. [5]
70-D	Oakland	16	1925	6.4	20	8	280+04—383+56 501+37—553+98	¾-inch edge bars and special [2]	do	Edge bars continuous	do	Deformed metal plate. Tie bars, 6-foot spacing.
70-D [3]	do	16	1932	10.4	10	10	280+04—383+56 501+37—553+98	Reinforced throughout with 60-pound wire fabric.	1 inch at variable spacing.	None	do	None. [4]
131-C	do	24	1930	8.6	20	10	122+50—173+43 285+23—334+65 478+56—531+49	Part of pavement reinforced [2]	1 inch at 100 feet	do	do	Deformed metal plate. Tie bars 40-inch spacing.
97-A	do	10	1924	4.7	20	9-7-9	250+20—346+49 366+27—429+26	None	¾ inch at 100 foot	do	do	Deformed metal plate. Tie bars, 5-foot spacing.
97-A [3]	do	10	1928	4.7	20	10-8-10	250+20—346+49 366+27—429+26	Part of pavement reinforced [2]	1 inch at 100 feet	do	do	None. [5]
41-R	do	10	1931	10.0	40	10	715+00—769+00 917+00—970+00 1084+98—1138+00	Reinforced throughout with 60-pound wire fabric.	¾ inch at 50 feet	do	do	Bulkhead. Butt joint. [2]
150-D [3]	Genesee	10	1932	4.0	20	10	295+40—375+84 419+27—498+89	do	1 inch at variable spacing.	do	do	None. [5]
146-B	Ingham	127	1925	4.0	20	9-7-9	326+57—484+79	None	do	do	do	Deformed metal plate. Tie bars, 5-foot spacing.
187-E	Branch	112	1918	1.4	16	6-8-6	1017+25—1066+68	do	1 inch at variable spacing.	do	do	None.

[1] A single dimension indicates uniform thickness.
[2] See table 2.
[3] Widening.
[4] Single lane abutted against 1 side of old pavement. No connection between old and new pavements.
[5] Single lane abutted each side of old pavement. No connection between old and new pavements.

TABLE 2.—*Special design features of various projects*

Federal-aid project No.	Special design features
100-A	¾-inch diameter edge bars, continuous across the joints: 111+20—340+00 Special reinforcement. Longitudinal steel; ten ½-inch diameter round at variable spacing: 66+00—111+20
101	¾-inch diameter edge bars, continuous across joints: 74+00— 96+00; 126+00—199+54; 337+80—445+00 Special reinforcement. Longitudinal steel; ½-inch diameter round, 24-inch spacing. Transverse steel; ½-inch diameter round, 28-inch spacing: 0+00— 8+50; 15+00— 74+00; 96+00—126+00 Longitudinal steel; ½-inch diameter round, 36-inch spacing. Transverse steel; ½-inch diameter round, 15-inch spacing: 8+50—15+00
139-E	The longitudinal joint at the center of the 40-foot pavement is of the straight-butt type. Tie bars at intervals of 20 inches were placed across this joint on the fill sections. No tie bars were used elsewhere. The longitudinal joints 10 feet from each edge of the pavement are of the weakened-plane type. No tie bars were used across these joints.
91	22-foot widening abutting old pavement. Longitudinal joint in the widening is of the weakened-plane type. No tie bars were used across this joint.
73-A	Load transfer provided across expansion joints in new pavement.
50-A	Special reinforcement in old pavement. Transverse steel; ¾-inch round, 12½-foot spacing.
70-D	Special reinforcement in old pavement. Longitudinal steel; ½-inch round, 6, 42, and 78 inches from free edges. Transverse steel; ½-inch diameter round, 15-inch spacing: 284+25—284+70; 287+00—295+00; 298+50—299+25; 316+25—316+55; 328+25—328+55; 334+00—344+00; 347+20—347+50; 365+60—365+90; 393+20—393+50; 412+60—412+90; 435+95—436+25; 444+00—465+00; 527+00—527+30; 535+40—535+70; 553+60—553+90; 515+70—516+72; 420+50—422+50; 465+00—494+00; 614+00—616+00
131-C	Special reinforcement. Wire fabric (60 pounds per square): 77+80— 84+07; 84+12—104+00; 135+00—144+00; 156+00—166+00; 310+00—336+05; 366+50—422+00; 543+00—559+00; 598+00—607+00; 617+00—627+50
97-A	Special reinforcement in widening. Wire fabric (60 pounds per square): 91+88—101+90; 199+44—207+00; 214+51—223+50; 278+18—281+36
41-R	Tie bars across longitudinal joints. ½-inch diameter round, 48 inches long, 40-inch spacing: 664+50—669+50; 674+50—681+00; 689+50—696+00; 699+50—728+00; 733+00—735+50; 738+00—739+50; 742+50—767+00; 791+50—809+50; 812+50—833+00; 837+00—847+00; 850+00—857+50; 870+50—887+00; 892+00—918+00; 928+00— 995+00; 1000+00—1013+00; 1018+00—1057+00; 1061+00—1070+50; 1082+50—1100+00; 1104+50—1108+00; 1110+50—1115+50; 1119+00—1145+00; 1152+00—1157+00; 1163+50—1166+50; 1170+00—1173+50; 1177+00—1191+28

REPRESENTATIVE SECTIONS OF PROJECTS EXAMINED IN DETAIL

The majority of the projects were of such length that it was impracticable to make a detailed study of the entire project. For this reason, 3 representative miles were selected for study on all projects exceeding 3 miles in length. Where the total length of the project was less than 3 miles, the entire pavement was examined. The combined total length of all of the projects studied is approximately 110 miles. Of this, 46 representative miles were actually surveyed in detail.

Certain subgrade data had been collected on some of the projects prior to this survey. These were supplemented by additional data obtained in a subgrade survey made as a part of the study. A complete subgrade survey was made on all projects where there had been no earlier survey. The data obtained permitted the general classification of the soils encountered but gave no indication of the density or of the moisture content of the subgrade beneath the pavement. On some of the projects several crack surveys had been made previously and, in such cases, the earlier crack surveys were simply brought up to date. On the remainder of the projects a complete crack survey was made for the section being studied.

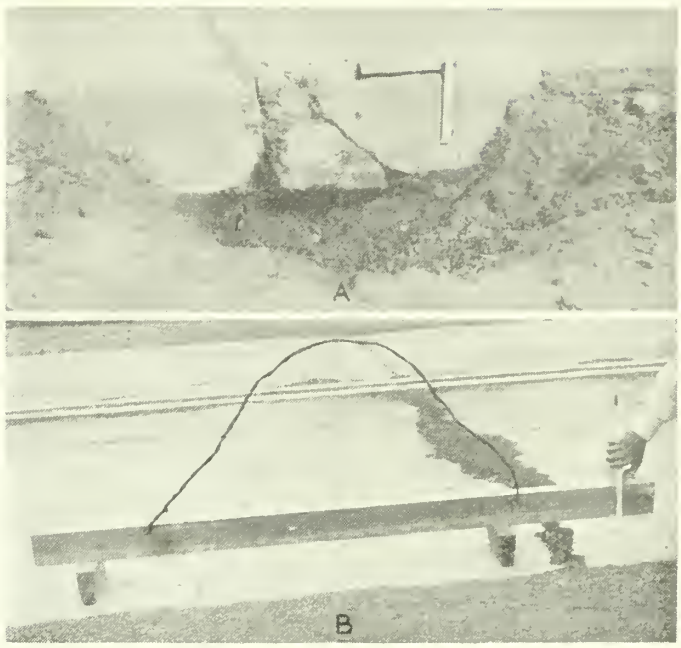


FIGURE 3.—A, SHOULDER REMOVED FOR INSPECTION OF EXPANSION JOINT; B, DEVICE FOR MEASURING FAULTING AT JOINTS AND CRACKS.

On all projects a detailed study was made of the joints in the course of which the following observations were made: (1) The width of opening of the joints and whether these openings were clear or filled; (2) The differences in elevation of the pavement surface on the two sides of the joint. These measurements were made at certain cracks, also; (3) The general condition of the joint, that is, whether or not it was spalled, broken, etc.

While the main object of the investigation was to study joints, it was thought desirable to make, in addition, a rather close examination of the general condition of the pavements and this developed a considerable amount of information not directly connected with joints.

Figure 3, A shows the method of exposing the joints for study. It was not practical to expose all of the joints as shown, but this was the general procedure followed. Some joints were examined in more detail than others.

Since it was desired to measure the difference in elevation or faulting at a large number of joints and cracks, it was necessary to devise a method permitting determination to be made quickly yet accurate enough for the purpose. Great precision was not justified because of the roughness of the concrete surfaces. The device used is shown in figure 3, B. The blocks that rested on the concrete were 4 feet apart and were always placed on the slab over which traffic approached the joint. The block nearest the joint was placed a foot or so back from it so that the local irregularities at the joint would not affect the measurements. Also the faulting measurements were made 4 or 5 inches from the joint for the same reason. The differences in elevation with respect to the arbitrary datum thus established were measured to the nearest one-sixteenth inch

TABLE 3.—*Summary of traffic survey data* [1]

Federal-aid project No.	Average daily traffic (all vehicles)	Average daily commercial vehicles								
		Number of vehicles	Average number of wheels per vehicle	Total number of wheel loads	Wheel loads 4,000 pounds or greater	Wheel loads 6,000 pounds or greater	Wheel loads 9,000 pounds or greater	Total wheel loads 4,000 pounds or greater	Total wheel loads 6,000 pounds or greater	Total wheel loads 9,000 pounds or greater
					Percent	*Percent*	*Percent*			
100-A	3,337	476	4.74	2,256	27.92	10.14	0.61	630	229	14
113-B	2,895	420	5.08	2,134	36.38	19.28	1.30	776	411	28
116	3,118	355	4.59	1,629	22.21	7.64	.66	362	124	11
101	2,691	673	5.14	3,459	32.39	16.45	2.02	1,120	569	70
91, 139-E	4,972	1,100	5.02	5,522	33.04	16.04	.43	1,824	886	24
89-D	2,650	439	4.54	1,993	30.03	14.00	1.85	598	279	37
95-DF	2,820	700	4.93	3,451	32.24	15.99	.97	1,113	552	33
73-A	7,622	1,884	5.12	9,646	38.73	21.63	1.27	3,736	2,086	122
50-A	8,634	1,990	5.12	10,180	38.73	21.63	1.27	3,943	2,202	129
70-D	6,420	887	4.80	4,258	25.41	12.52	1.97	1,082	533	84
131-C	2,635	650	5.20	3,380	30.91	14.75	1.70	1,045	499	57
97-A	9,828	1,695	5.02	8,509	29.00	15.17	1.23	2,468	1,291	105
41-R	6,926	1,028	5.02	5,161	29.00	15.17	1.23	1,497	783	63
150-D	6,124	700	4.69	3,283	28.16	14.42	1.38	924	473	45
146-B	3,670	389	4.35	1,692	21.65	8.22	.27	366	139	5
187-E	3,400	707	4.93	3,486	32.24	15.99	.97	1,124	557	34

[1] The basic data were collected by the Michigan Highway Planning Survey between January 1936 and January 1937.

TABLE 4.—*Subgrade classification*

Federal aid project No.	Mile surveyed	Subgrade soil group	Soil series	Special subgrade conditions within the area surveyed
100-A	1	A-6	Maimi and Conover	Subgrade rests on unstable peat between stations 162 and 167.
	2	A-6	do	
	3	A-6	do	
113-B	1	A-6	Napanee with small areas of Miami and Brookston	A sand and gravel subbase with stone or gravel bleeders was placed between stations 1427+20 and 1455; 1628+80 and 1660.
	2	Mainly A-6	Napanee and Miami with small areas of Coloma	
	3	Mainly A-6	Miami and Conover with small areas of Brookston	
116	1 and 2	A-3	Firdst ¾ mile Fox and Warsaw (outwash area). Remaining 1¼ miles uniform Bellefontaine and Coloma.	A 4-inch sand subbase was placed between stations 171 and 173+50; 177+50 and 180; 190+97 and 225.
	3	A-3, A-6	Coloma to Isabella	
101	1	Mainly A-3	Variable—Plainfield, Berrien, Brookston, Coloma	The water table is near the surface between stations 111 and 133 and an 8-inch gravel subbase was placed between stations 129 and 134+61.
	2	A-3	Mianly Fox and Plainfield	
	3	A-3	Fox and Plainfield	
91 and 139-E	1	A-3	Bridgman, Newton, and Plainfield	The water tahle is near the surface between stations 12 and 15. The fill settled between stations 145 and 148.
	2 and 3	A-3	Variable—Plainfield, Berrien, Ottawa, Saugatuck	
89-D	1	A-3	Fox	
	2	A-6	Hillsdale	
	3	A-3, A-6	Hillsdale except last 1,500 feet which is Oshtemo and Plainfield.	
95-DF	1	A-3, A-6	Mainly Miami with some Coloma	The surface relief is generally rolling with deep cuts and high fills.
	2	A-3	Mainly Bellefontaine and Coloma	
	3	A-3	Variable—mainly Fox with some Napanee, Plainfield, and Hillsdale.	
73-A	First 1.5	A-6, approaching A-7	Brookston, Brown Napanee	This pavement is on an old lake hed and the surrounding area is very flat. A large part of the old pavement is laid on a 6- to 10-inch well-drained sand and gravel subbase.
	Second 1.5	A-3, A-6	Mainly Brown Napanee with some Brookston and Hillsdale.	
50-A	1 and 2	A-6, approaching A-7	Brown Napanee	This pavement is in the same area as project 73-A and the suhgrade is of the same type except no suhhase was used.
	3	do	Mainly Brown Napanee with small area of Genesee	
70-D	1, 2, and 3	A-6	Miami and Conover	The fill has settled between stations 364 and 366.
131-C	1	A-3	Newton, Gilford, Berrien, and Ottawa	Tile edge drains hackfilled with gravel were placed between stations 133 and 144; 158 and 165; 517 and 524.
	2	A-6	Napanee, Brookston, Allendale, and Conover	
	3	A-3, A-6	Miami and Bellefontaine	
97-A	First 1.8	A-3	Mainly Fox and Oshtemo	The fill has settled over an unstahle peat area hetween stations 278 and 281.
	Second 1.2	A-3 predominant	Mainly Fox and Oshtemo with some Miami and Bellefontaine.	
41-R	1	A-3, A-6	Miami and Bellefontaine	The fill has settled tc some degree between stations 761+50 and 763+25; 935+50 and 937+25.
	2	A-6	Miami	
	3	A-3, A-6	Mainly Miami with some Bellefontaine	
150-D	First 1.5	A-6	Miami, Conover, Brookston with some Bronson	
	Second 1.5	A-3, A-6	Variahle—Ottawa, Berrien, Brady, Bronson, Newton, Allendale, Brookston, and Conover.	
146-B	1, 2, and 3	A-3, A-6	Variahle—Miami, Conover, Hillsdale, Newton, Maumee, Bellefontaine, and Oshtemo.	A large part of this pavement is on a sandy fill over poorly drained muck. The pavement has settled between stations 344+10 and 347+21; 398+10 and 401+10, 409+82 and 410+93; 419+70 and 419+90.
187-E	1	A-1, approaching A-3	Warsaw	

in the manner shown. The measured differences are not necessarily true differences in elevation but are, rather, the departure at a given point on the slab beyond the joint from the datum established by the surface of the approach slab. The measurements were made perpendicular to the surface of the pavement rather than in a true vertical direction. It is thought that this measured difference is best described as "faulting" at the joint and this term will be used hereafter. Measurements of faulting were made at a sufficient number of joints and cracks so that errors caused by the occasional irregularities of the concrete surface would be largely eliminated in the averages.

General observations were made of the condition of the joints, cracks, and the condition of the pavement as a whole and a number of photographs were made.

A summary of the traffic data on the various sections is given in table 3. The average daily traffic is shown in the second column, while the average daily commercial traffic is shown in the third column. In the last three columns the commercial traffic is divided into three wheel-load weight classes.

Certain subgrade survey data for the different sections are shown in table 4 (*4*).[2] Michigan uses the "soil series" method of soil classification in highway work.

[2] Italic numbers in parenthesis refer to bibliography, p. 205.

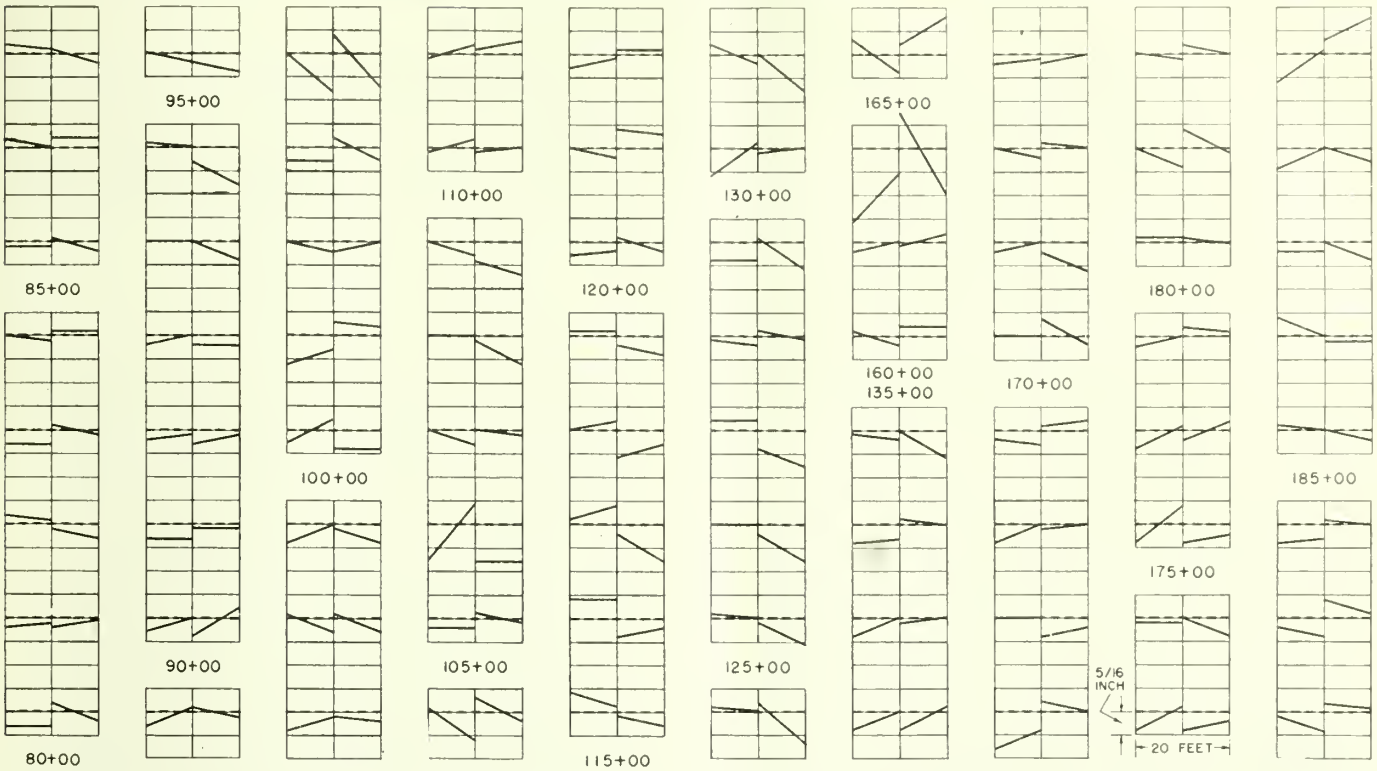


FIGURE 4.—TYPICAL FAULTING MEASUREMENTS SHOWING THE VERTICAL POSITION OF THE SLAB BEYOND THE JOINT WITH RESPECT TO THE APPROACH SLAB IN THE DIRECTION OF TRAFFIC.

With the following exceptions, the concrete in the pavements examined contained silicious gravel as the coarse aggregate: (1) The widening on project 50–A; (2) the original pavement of project 73–A; and (3) in part of the pavement of project 95–DF. In these three sections crushed stone coarse aggregate was used.

GENERAL CONDITION OF INDIVIDUAL PROJECTS DESCRIBED

Project 100–A.—This two-lane pavement was 13 years old at the time of the survey. In general, slab length varied between 16 and 28 feet and over the whole project averaged less than 20 feet. The pavement was rough over small areas where the subgrade had settled. In general, there was no unusual amount of spalling or breakage at the transverse joints. A large percentage of the cracks were open and many of them were badly spalled and disintegrated.

Project 113–B.—This two-lane pavement was 12 years old at the time of the survey. The slab length varied generally between 12 and 18 feet and averaged 14 feet. The joints were generally in good condition and there was only a moderate amount of spalling at the cracks.

Project 116.—This two-lane pavement was 14 years old at the time of the survey. The average slab length in the first 2 miles, with the sand and gravel subgrade, was approximately 18 feet, while that in the third mile, where the less desirable subgrade is found was approximately 16 feet. The 4-inch sand subbase, in certain areas, appeared to have had little, if any effect on the amount of cracking.

The joints were generally in good condition. The cracks in the first 2 miles were closed and very little spalling had occurred. The cracks in the third mile were open slightly and a greater degree of spalling had taken place.

Project 101.—This two-lane pavement was 14 years old at the time of the survey. The water table was relatively close to the surface in the first mile and this appears to have caused considerable cracking from frost action. The average slab length in this mile ranged generally from 10 to 20 feet. The gravel subbase and special reinforcement in parts of this mile appear to have had little if any effect on the amount of cracking. The average slab length in the second and third miles was greater than that in the first mile, ranging generally from 15 to 35 feet. A large percentage of the cracks in the first mile were open and badly spalled, but generally the cracks in the second and third miles were in better condition. There was a moderate amount of spalling at the joints. Considerable scaling, especially in the first mile, had occurred at the intersection of the cracks and longitudinal joint.

Project 139–E.—This four-lane reinforced pavement was 7 years old at the time of the survey. It was in much better condition than most of the other pavements examined. The slab lengths varied generally between 40 and 80 feet and the average slab length was 47 feet. The great majority of the cracks were closed and practically no spalling or disintegration had occurred at the joints, cracks, or other parts of the pavement. While the fact that this pavement is only 7 years old is no doubt responsible to some degree for its good condition, it is probable that the favorable subgrade conditions and the presence of welded fabric reinforcement have also been important contributing factors.

Project 91.—This was originally a two-lane pavement built in 1923. Two additional lanes were added at one side of the old pavement in 1931. The original pavement was plain concrete and the new pavement was reinforced with welded fabric. Both pavements were in better condition than the majority of the other pavements of same age, and this was especially noticeable on the newer pavement. The average slab lengths of the

TABLE 5.—*Summary of data*

Federal-aid project No.	Cross section	Year built	Average 24-hour traffic, 1936	Number of heavy wheel loads, average daily, 1936 (commercial)			Original slab length	Average slab length, 1938	Number of corner breaks at expansion joints per lane mile	Expansion joints where maximum difference in elevation of slabs was [2]—			Cracks [1] where maximum difference in elevation of slabs was [2]—			Number of transverse cracks, per lane mile	
				4,000 pounds or above	6,000 pounds or above	9,000 pounds or above				¼ inch or more	⅛ inch to 3/16 inch	1/16 inch [3]	¼ inch or more	⅛ inch to 3/16 inch	1/16 inch [3]	Within 8 feet of expansion joints	Within 11 feet of expansion joints
	Inches						*Feet*	*Feet*		*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*		
100–A	9–7–9	1925	3,337	630	229	14	94	18	1	36	42	18	4	31	46	6	15
113–B	9–7–9	1926	2,895	776	411	28	98	14	1	47	35	14	8	32	41	14	35
116	7	1924	3,118	362	124	11	47	17	1	27	49	18	2	24	32	7	22
101	8, 9–7–9	1924	2,691	1,120	569	70	108	18	1	42	41	14	6	38	36	6	18
139–E	9	1931	4,972	1,824	886	24	92	47	0	11	42	31				0	1
91	8	1923	4,972	1,824	886	24	358	24	0				5	41	38		
91 [4]	9	1931	4,972	1,824	886	24	93	66	0	15	50	22				1	2
89–D	8	1924	2,650	598	279	37	88	17	1	22	48	25	2	38	39	3	12
95–D, F	9–7–9	1926	2,820	1,113	552	33	90	23	0	25	45	20	7	50	27	7	21
73–A	10–8–10	1925	7,622	3,736	2,086	122	82	15	0	30	42	16	1	37	40	14	28
73–A [4]	10–8–10	1935	7,622	3,736	2,086	122	27	26	0	2	17	46				1	2
50–A	9	1923	8,634	3,943	2,202	129	133	18	3	48	33	10	8	42	23	5	7
50–A [4]	10	1933	8,634	3,943	2,202	129	98	37	0	48	40	10				6	10
70–D	8	1925	6,420	1,082	533	84	175	16	0	32	38	18	8	44	33	5	8
70–D [4]	10	1932	6,420	1,082	533	84	92	38	0	21	46	23	0	38	36	1	2
131–C	10	1930	2,635	1,045	499	57	93	25	0	23	43	25	5	26	42	1	3
97–A	9–7–9	1924	9,828	2,468	1,291	105	91	17	1	36	42	16	7	38	33	9	23
97–A [4]	10–8–10	1928	9,828	2,468	1,291	105	99	20	0	48	40	7	2	34	34	5	12
41–R	10	1931	6,926	1,497	783	63	49	25	0	38	39	14	2	51	35	1	2
150–D [4]	10	1932	6,124	924	473	45	99	45	0	36	42	14	1	61	33	0	1
146–B	9–7–9	1925	3,670	366	139	5	90	17	6	36	40	16	5	43	38	9	18
187–E	6–8–6	1918	3,400	1,124	557	34	72	30	0	50	33	11	15	50	25	4	11

[1] Those at which measurements were made.
[2] In determining these percentages a joint or crack in a single pavement lane was considered to be a unit.
[3] Faulting less than 1/16 inch not included.
[4] Widening.

old and new pavements at the time of the survey were 25 and 66 feet, respectively. The cracks in the old pavement were open and badly spalled, but the joints and cracks in the new pavement were in good condition. It is thought probable that the favorable subgrade conditions and the presence of the distributed reinforcement were largely responsible for the limited amount of transverse cracking.

Project 89–D.—This two-lane pavement was 14 years old at the time of the survey. The slab lengths varied generally between 16 and 20 feet, but there was one area, notably between stations 1510 and 1522, where the average slab length was greater than 20 feet. The majority of the transverse cracks were open and a considerable amount of spalling and disintegration had occurred in their vicinity. The joints were in good condition except for a moderate amount of spalling at the intersection of the longitudinal and transverse joints.

Project 95–DF.—This two-lane pavement was 12 years old at the time of the survey. Much of it was badly scaled and had been covered with a thin film of bituminous material. It was necessary, for this reason, to select for the survey 3 miles which were in better than average condition. Slab lengths throughout the first mile were uniformly low and the average for the whole mile was 14 feet. Slab lengths in the second and third miles varied generally between 20 and 50 feet and averaged approximately 29 and 26 feet, respectively. A considerable amount of spalling and disintegration had occurred at the joints and cracks.

Project 73–A.—This is a three-lane pavement. The first two lanes were of plain concrete, built in 1925. The third lane was built 10 years later and is reinforced. Slab lengths on the old part of the pavement varied generally between 15 and 20 feet and averaged approximately 15 feet. There had been little cracking in the new part of the pavement up to the time of the survey, probably because of the use of joints at approximately 25-foot intervals.

The new part of the pavement is in good condition in every respect. In contrast, the joints and cracks in certain parts of the old pavement are badly spalled and distintegrated.

Project 50–A.—This is a four-lane pavement. The two inside lanes were built in 1923 and the two outside lanes were added in 1933. The old pavement is reinforced throughout with a special reinforcement, as indicated in tables 1 and 2. The newer pavement is reinforced throughout with a 60-pound welded fabric. The slab length of the older pavement varied generally between 14 and 20 feet and averaged approximately 17 feet. The slab length of the newer pavement varied generally between 25 and 80 feet and averaged approximately 37 feet. The newer part of the pavement was apparently in much better condition than the old pavement had been at the same age, possibly because of differences in the design and manner of placing the reinforcement in the two pavements.

Spalling had occurred at the joints and cracks in the old pavement and on one short section a considerable amount of scaling in the vicinity of the cracks was noted. The joints and cracks in the new pavement were in good condition.

Project 70–D.—The section included in this survey was originally a two-lane pavement built in 1925 to which a third lane was added in 1932.

The slab length of the older pavement varied generally between 12 and 20 feet and averaged approximately 16 feet. The slab length of the newer pavement varied generally between 20 and 75 feet and there were a number of slabs which exceeded 80 feet in length. The average slab length of the newer pavement over the entire section surveyed was approximately 38 feet at the time of the survey.

The general condition of much of the old pavement was poor, with spalling and scaling prevalent along the joints and cracks. The joints and cracks in the new part of the pavement were in good condition.

Project 131–C.—The section surveyed is a two-lane pavement built in 1930. The slab length in the first mile varied generally between 25 and 50 feet and averaged approximately 33 feet. The slab lengths in

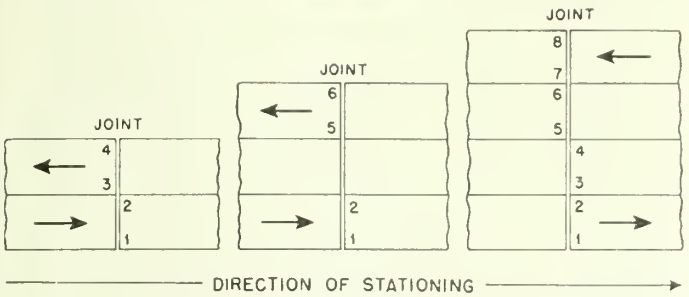


FIGURE 5.—POINTS AT WHICH FAULTING MEASUREMENTS WERE MADE ON TWO-, THREE-, AND FOUR-LANE PAVEMENTS.

FIGURE 6.—EXPANSION JOINT FAULTED APPROXIMATELY 1 INCH WITH THE SLAB BEYOND THE JOINT DOWN.

the second and third miles varied generally from 15 to 30 feet and averaged approximately 21 feet. The greater slab length in the first mile was probably due to the more favorable subgrade conditions. The cracks and joints in this pavement were in good condition.

Project 97–A.—This is a four-lane pavement, the two center lanes of which were built in 1924 and two outside lanes were added in 1928. A great deal of cracking had occurred in this pavement which is surprising in view of the favorable subgrade conditions. The average slab length for that part of the pavement built in 1924 was approximately 17 feet, while that for the newer pavement built in 1928 was approximately 20 feet.

The concrete in the vicinity of the joints was in most cases in fairly good condition although there was spalling and scaling in the vicinity of many of the cracks in the older part of the pavement. The cracks in the newer part of the pavement were in better condition.

Project 41–R.—This is a four-lane reinforced pavement which was 7 years old at the time of the survey. The pavement was in good condition. The slab length varied generally between 20 and 30 feet although there were a number of sections where it exceeded 30 feet. The average slab length of the entire portion of the pavement surveyed was approximately 25 feet. The cracks were closed and both the joints and cracks were in good condition.

Project 150–D.—The section surveyed consisted of two 10-foot lanes constructed, one on each side of an old 20-foot pavement, which has been resurfaced. The new part of the pavement was reinforced. It was 6 years old at the time of the survey, and was in good condition. There were many long slabs, the average slab length exceeding 40 feet. The cracks were closed and both the joints and the cracks were in excellent condition.

Project 146–B.—The section studied is a two-lane pavement which was 13 years old at the time of the study. The slab length varied generally between 10 and 20 feet, the average slab length being approximately 17 feet. Some of the shorter slabs are located in areas where fills have settled or where frost action had broken the pavement. The cracks were open and a moderate amount of spalling had developed at both the joints and the cracks. The surface of the pavement may be described as being in fairly good condition.

Project 187–E.—The section surveyed is a 16-foot pavement built in 1918 and originally identified as State Reward Project No. 6269. The cross section is 6–8–6 inches and there was no longitudinal joint although a longitudinal crack has developed throughout most of the pavement.

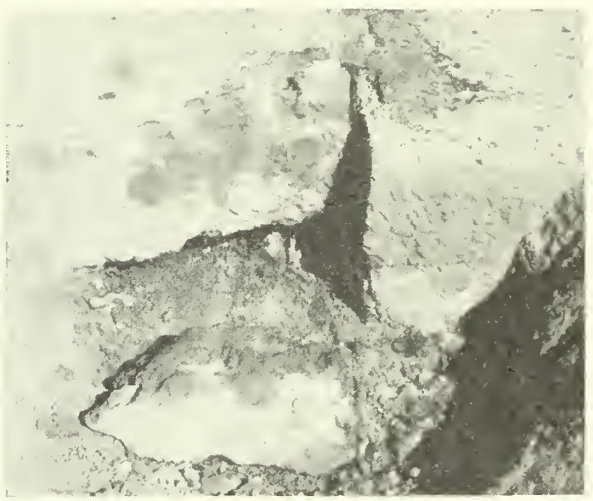


FIGURE 7.—SMALL COMPRESSION CORNER FAILURE AT A CLOSED EXPANSION JOINT.

This pavement was in good condition for its age. The average slab length was approximately 30 feet. The relatively small amount of transverse cracking that has developed may be attributed to the favorable subgrade. The dense shade which exists along most of the section may have had some influence in limiting cracking.

The joints were generally in good condition. Many of the cracks, however, were open and considerable spalling had occurred.

Certain defects were observed quite generally in all sections studied but to avoid repetition they were not mentioned in the preceding discussion. These were: (1) Faulting at joints and cracks; (2) closure of the expansion joints and the presence of foreign material in the joints and cracks; and (3) breakage and disintegration of the concrete in the vicinity of joints and cracks.

FAULTING AT JOINTS AND CRACKS

Typical data showing the magnitude and direction of faulting at a number of the joints on a two-lane pavement are shown in figure 4. The points at which faulting was measured on two-, three-, and four-lane pavements and the numbers identifying the points are shown in figure 5. Figure 4 indicates the vertical position at the different points on the slab beyond the

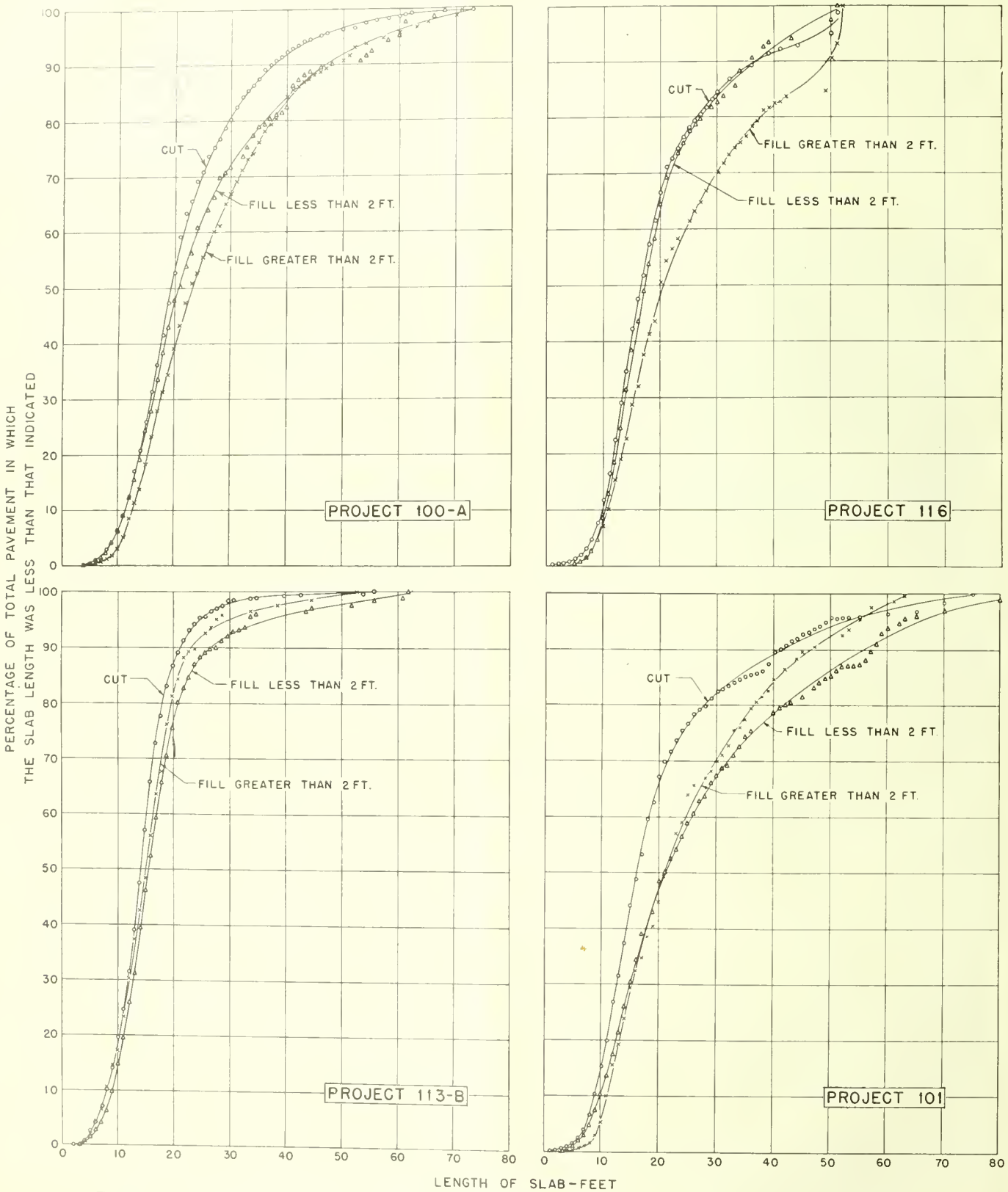

PERCENTAGE OF TOTAL PAVEMENT IN WHICH THE SLAB LENGTH WAS LESS THAN THAT INDICATED
CUT
FILL LESS THAN 2 FT.
FILL GREATER THAN 2 FT.
PROJECT 100-A
PROJECT 116
PROJECT 113-B
PROJECT 101
LENGTH OF SLAB-FEET

FIGURE 8.—PERCENTAGE DISTRIBUTION OF SLAB LENGTHS IN CUTS AND ON FILLS ON FOUR PROJECTS.

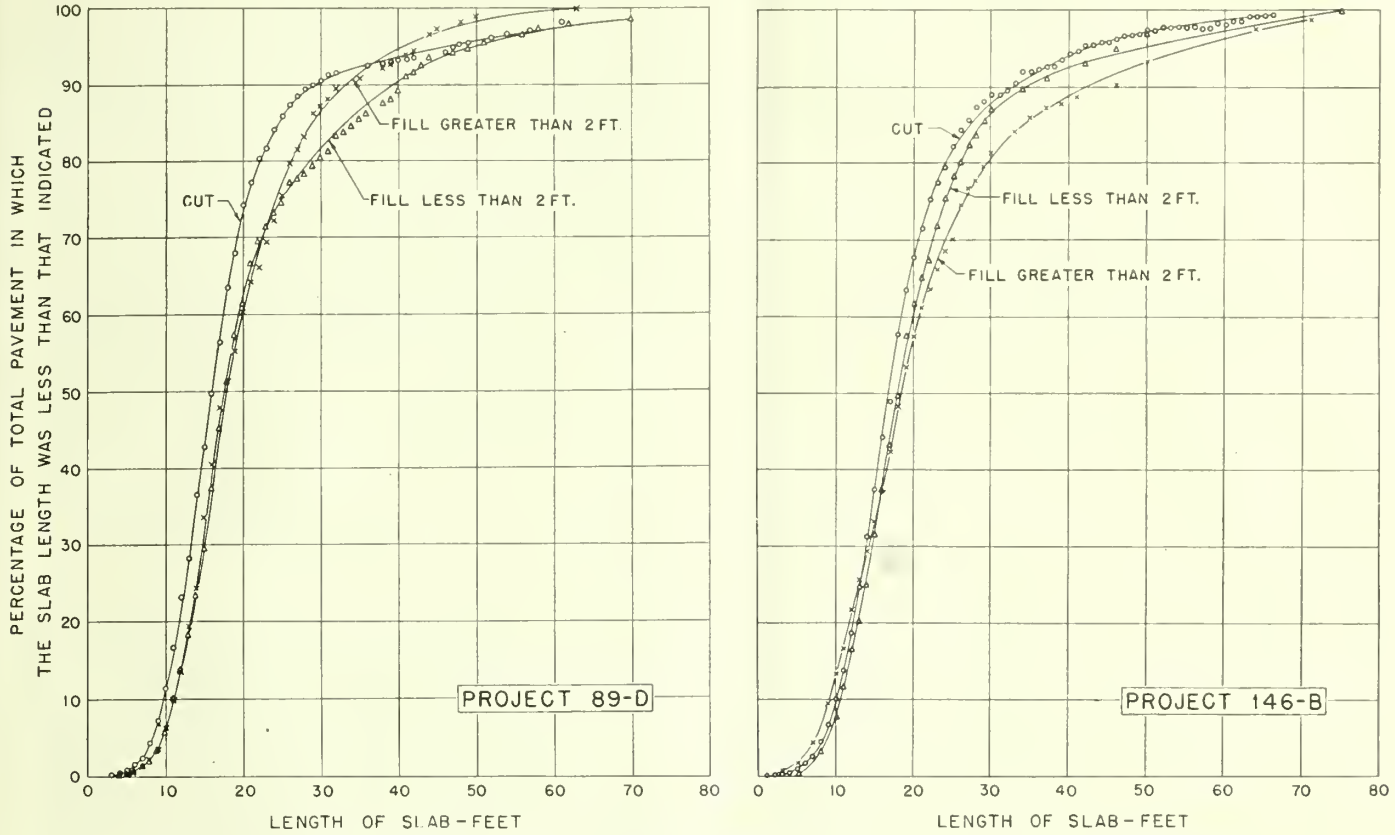

PERCENTAGE OF TOTAL PAVEMENT IN WHICH THE SLAB LENGTH WAS LESS THAN THAT INDICATED
CUT
FILL GREATER THAN 2 FT.
FILL LESS THAN 2 FT.
PROJECT 89-D
LENGTH OF SLAB - FEET
CUT
FILL LESS THAN 2 FT.
FILL GREATER THAN 2 FT.
PROJECT 146-B
LENGTH OF SLAB - FEET

FIGURE 9.—PERCENTAGE DISTRIBUTION OF SLAB LENGTHS IN CUTS AND ON FILLS ON TWO PROJECTS.

joint with respect to the approach slab. Thus, a minus value at a given point indicates that, at this point, the slab beyond the expansion joint is below the approach slab. A positive value indicates the opposite condition.

A majority of the joints on this project were faulted. The magnitude of the faulting was generally greater at the free edges of the pavement than along the connected longitudinal joints, but at certain places a considerable degree of faulting was present near the longitudinal joint.

There appears to be no uniformity in the direction or manner of the faulting. In many cases the slab beyond the joint was below the approach slab at the free edges of the pavement, while in others the opposite condition was found. The same was true of the two points adjacent to the longitudinal joint. As a general rule, however, the slab beyond the joint was below the approach slab where the faulting was severe, that is, where the relative displacement was one-half inch or more. Such a joint is illustrated in figure 6.

While some faulting was present at cracks, the displacement was much less than at expansion joints.

On all projects the degree of faulting at the expansion joints varied with the age of the pavement, the type of subgrade, etc., but the general pattern was approximately the same. The degree of displacement found at the joints will be discussed later in connection with table 5.

CONDITION OF JOINTS AND CRACKS

A large percentage of the expansion joints examined were found to be closed or partly closed. Whatever spaces remained between the slab ends were filled with hard, compressed soil. This condition had brought about a number of small compression corner failures of the type shown in figure 7. In many of the sections a large percentage of the transverse cracks were open and badly spalled and disintegrated.

A summary of the pertinent facts found in the study s given in table 5.

AVERAGE SLAB LENGTH OF CONCRETE PAVEMENTS DETERMINED FOR VARIOUS SUBGRADE AND TRAFFIC CONDITIONS

The data were examined to determine the effect of a number of variables on the average slab lengths of the various pavements as shown in table 5.

The effect of cut and fill on average slab length.—This relation is shown in figures 8 and 9, inclusive. These are typical distribution curves in which the abscissas indicate slab lengths while the corresponding ordinates represent the percentage of the total pavement length formed by slabs of that length or less.

The depth of fill used in the construction of these graphs is the average for the two sides of the pavement. While the depths of the fill at the edges of the pavement generally refer to the natural ground, they sometimes differ considerably from the actual depth of fill placed at the center of the pavement at the last construction, since a number of the pavements were built over old roadbeds and the depths of the fills were measured to the old roadbeds. The depths of the fills on the sides of the pavement were not, as a rule, influenced by the old road because the new surface was generally built wider than the existing road.

In interpreting the graphs, it should be emphasized that there are relatively few long slabs in many of the pavements from which the data were obtained. Thus the upper parts of these curves represent only small samples of pavements. Also, on some of the projects the total length of pavement, on fills greater than 2 feet in height, was relatively small. These facts may ex-

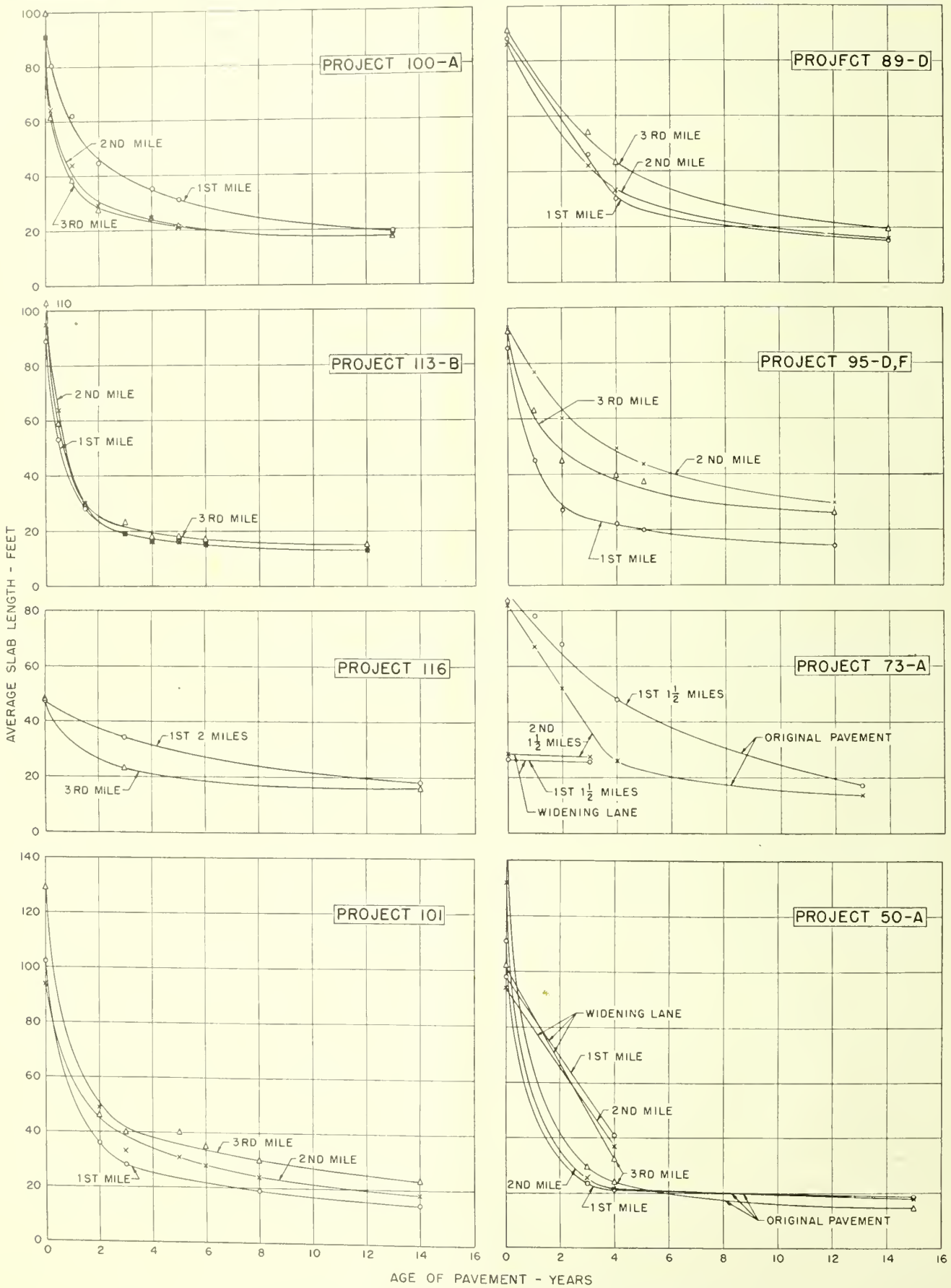


FIGURE 10.—CHANGE IN AVERAGE SLAB LENGTH WITH AGE OF PAVEMENT ON EIGHT PROJECTS.

plain certain inconsistencies in some of these graphs, especially in the upper ends of the curves.

The graphs are consistent in showing a greater slab length on fills than in cuts. The difference is not great for that half of the pavement lengths having the shortest slabs, but is of some size where slabs were longest. In some cases the slab lengths are greater on fills less than 2 feet in depth than on fills more than 2 feet in depth, while in other cases the opposite is true. The subgrade material on projects 100–A and 113–B is clay, while that on projects 116 and 101 is predominantly sandy. The subgrade material on projects 89–D and 146–B is variable.

In the cuts the percentage of pavement with an average slab length of 20 feet or greater is approximately the same for the three types of subgrade material. On the fills however the percentage is somewhat greater for pavement on sandy subgrade.

AVERAGE SLAB LENGTH OF CONCRETE PAVEMENTS AT DIFFERENT AGES

The average slab lengths at different ages for certain sections examined for which data from former crack surveys were available are shown in figures 10 and 11. On some of the newer pavements only one crack survey had been made and for these only the original slab lengths and slab lengths at the time of this survey can be shown.

The more recently built pavements (projects 73–A, 50–A, 70–D, and 146–B) shown in figures 10 and 11 are separate lanes 10 feet in width which have been placed along the side of older pavements. These lanes generally were laid on the same types of subgrades as the old pavements. Reinforcement has been incorporated more generally in the newer pavements than in the older ones and the effect of this reinforcement will be discussed later in the report.

Figures listed above consistently show that, for these pavements, transverse cracking greatly reduces the average slab length and the greatest reduction in slab length occurs within the first few years of life of the pavement. The majority of the transverse cracks developed during the first 4 years on these projects. All of the pavements tend to reach an ultimate average slab length between 15 and 20 feet.

The average slab lengths of the widening lanes on projects 50–A and 70–D are greater than those of the original pavement at the same age. The average slab length of the newer pavement on project 97–A is practically the same as that of the old pavement at the same age.

The average slab length of the new pavement on project 73–A is much less than that of the old pavement at the same age, for the reason that joints were placed at intervals of approximately 25 feet in the new pavement at the time it was built. Figure 10 (project 73–A) shows that very little cracking occurred during the first 3 years of the life of the new pavement, a condition that is undoubtedly due to the fact that the pavement was originally constructed in 25-foot slabs.

AVERAGE SLAB LENGTHS OF PAVEMENTS ON DIFFERENT SUBGRADE MATERIALS COMPARED

In figure 12 the average slab length for the separate miles within each section are shown grouped according to the general type of the subgrade material on which the pavement was laid. There are two general types of material represented, the clays, falling generally in the A–6 group and the sandy soils of the A–3 group.

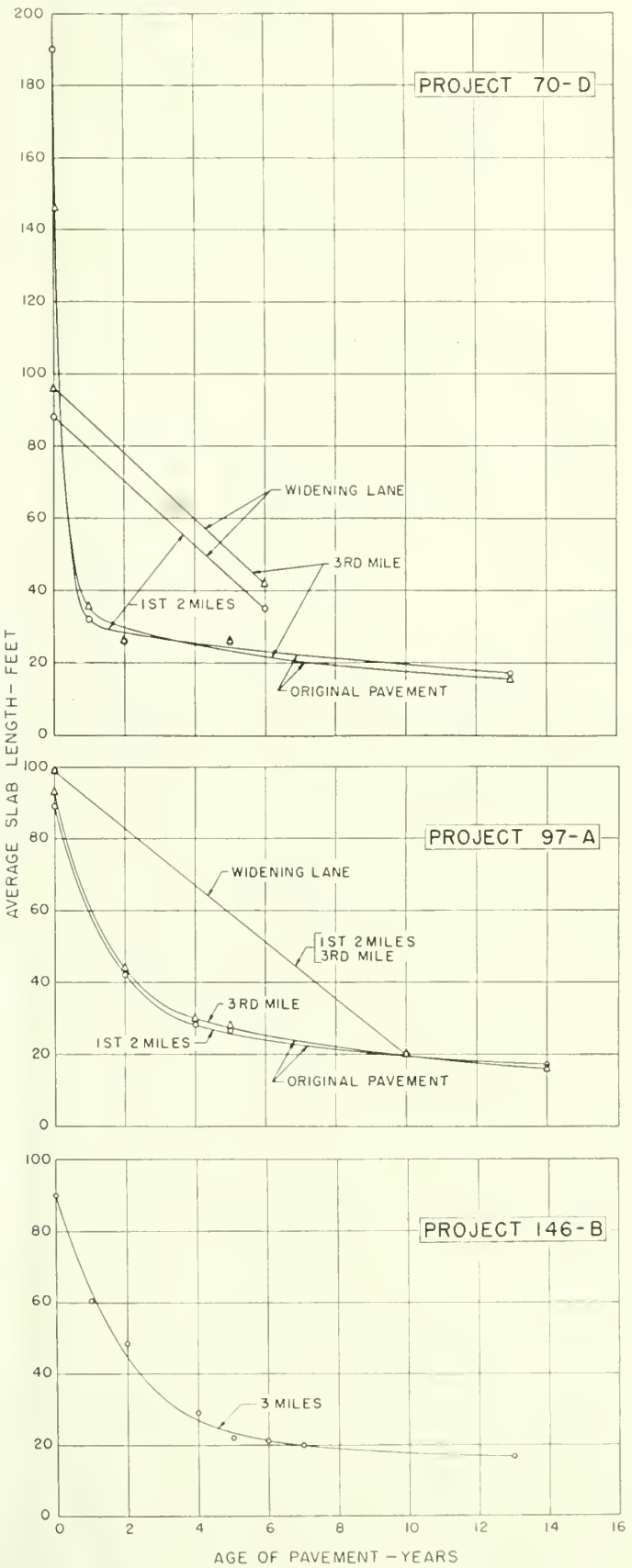


FIGURE 11.—CHANGE IN AVERAGE SLAB LENGTH WITH AGE OF PAVEMENT ON THREE PROJECTS.

In some cases the clay materials approach the A–7 group and the soil of one project (187–E) was classed as A–1, approaching A–3. In general, the sandy

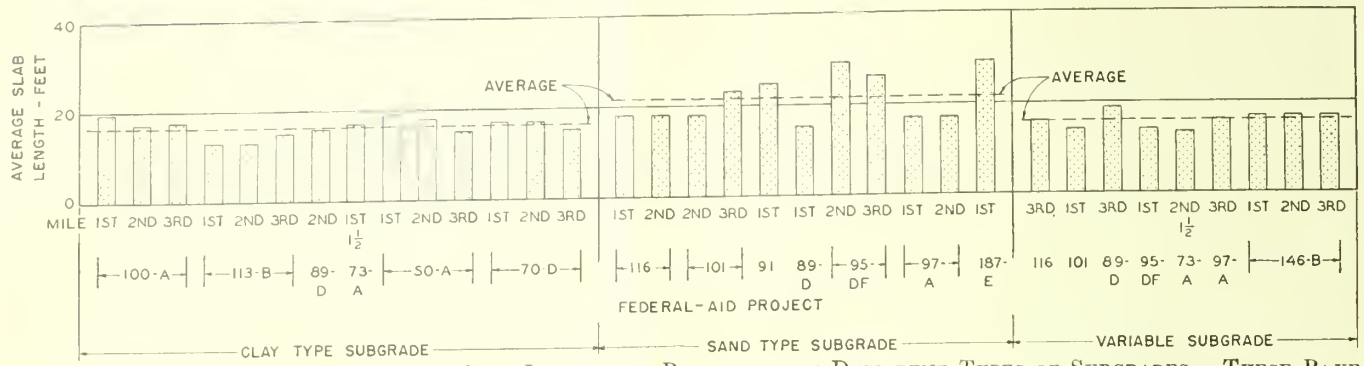

AVERAGE SLAB LENGTH - FEET
40
20
0
AVERAGE
AVERAGE
MILE 1ST 2ND 3RD 1ST 2ND 3RD 2ND 1ST 1ST 2ND 3RD 1ST 2ND 3RD 1ST 2ND 2ND 3RD 1ST 1ST 2ND 3RD 1ST 2ND 1ST 3RD 1ST 3RD 1ST 2ND 3RD 1ST 2ND 3RD
100-A 113-B 89-D 73-A 50-A 70-D 116 101 91 89-D 95-DF 97-A 187-E 116 101 89-D 95-DF 73-A 97-A 146-B
FEDERAL-AID PROJECT
CLAY TYPE SUBGRADE
SAND TYPE SUBGRADE
VARIABLE SUBGRADE

FIGURE 12.—COMPARISON OF THE AVERAGE SLAB LENGTHS OF PAVEMENTS ON DIFFERENT TYPES OF SUBGRADES. THESE PAVEMENTS RANGE IN AGE FROM 12 TO 15 YEARS, WITH THE EXCEPTION OF 187–E WHICH IS 20 YEARS OLD.

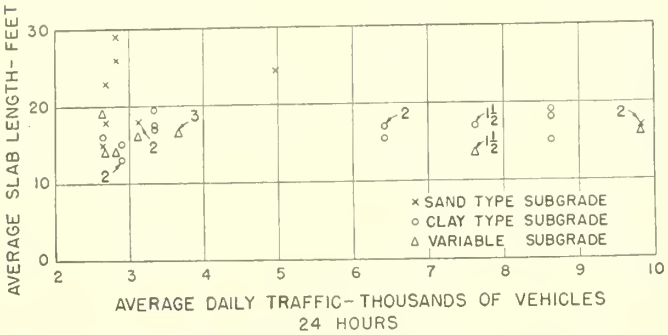


FIGURE 13.—COMPARISON OF AVERAGE SLAB LENGTHS ON PAVEMENTS WITH DIFFERENT AMOUNTS OF TRAFFIC. EACH POINT REPRESENTS 1 MILE EXCEPT WHERE INDICATED DIFFERENTLY. PAVEMENTS RANGE IN AGE FROM 12 TO 15 YEARS.

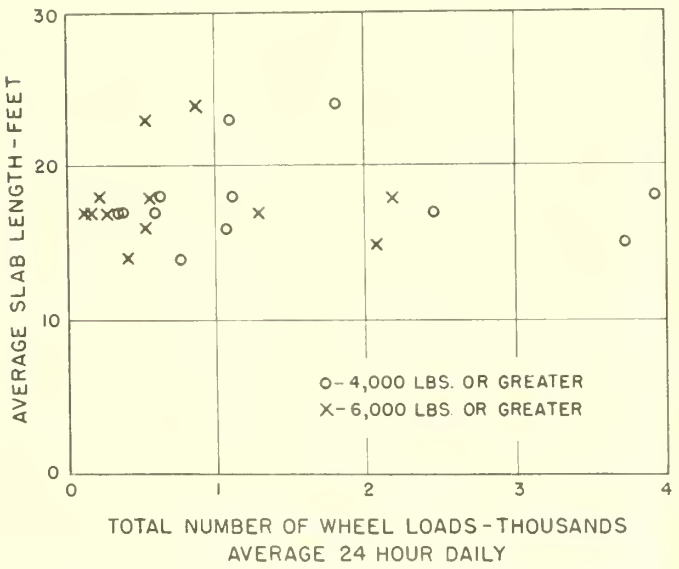


FIGURE 14.—COMPARISON OF AVERAGE SLAB LENGTHS ON PAVEMENTS HAVING DIFFERENT AMOUNTS OF HEAVY TRUCK TRAFFIC. EACH POINT REPRESENTS A SECTION OF PAVEMENT OF ABOUT 3 MILES. PAVEMENTS RANGE IN AGE FROM 12 TO 15 YEARS.

materials are considered to be better than the clays. The subgrade materials that are classified as variable are of the same general character as the clay or sandy subgrades but change from one type to the other, one or more times within the mile of pavement surveyed. No tests were made to determine the density and the moisture state of the subgrades under the various pavements studied. The very general nature of the survey and the considerable mileage of pavement involved made it impracticable to include such tests.

All of the pavements included in this comparison were between 12 and 15 years old, except that on project 187–E which was 20 years old.

Figure 12 shows an average slab length of approximately 16 feet on both the clay and the variable type subgrades, while that on the sandy material was approximately 21 feet. Thus the average slab length on the sandy soils was approximately 30 percent greater than that on the clay soils. It is noted that the average slab length for project 187–E was greater than that on any other project. This was probably the result of favorable subgrade conditions and the removal of some of the poorest sections of the pavement before the survey was made.

Sand and gravel subbases were placed on certain parts of projects 113–B, 116, 101, and 73–A (see table 4). It was found that the average slab length for the parts of the pavement laid over these subbases was approximately the same as those for other parts of the pavement. It may be assumed that these subbases were placed to correct some defect in the subgrade at the time the pavements were built and it appears that, in general, the pavements where the subbase was used were as satisfactory as those on the remainder of the subgrade.

Parts of some of the pavements in figure 12 had small amounts of special reinforcement. This reinforcement had not influenced the average slab length of the pavements sufficiently to affect the relations shown. The effect of reinforcement in general is discussed later in the report.

TRAFFIC VOLUME HAD NO IMPORTANT EFFECT ON SLAB LENGTH

Figures 13 and 14 show the relation between the average slab lengths of the various pavements and the amount of traffic they carry. The slab length and traffic data from which these graphs were constructed are shown in table 5. These traffic data were collected during 1936 after most of the cracking in the pavements had already occurred. Also a few of the surfaces were widened to three and four lanes some time after they were built and this has had an important effect on the traffic in a given pavement lane. Figure 13 indicates that there were a greater number of the longer slabs on the pavements with light traffic however, the longer average slab lengths were for pavements laid on the sandy subgrades.

Considering the data as a whole, it must be concluded from figure 13 that, so far as these surfaces are

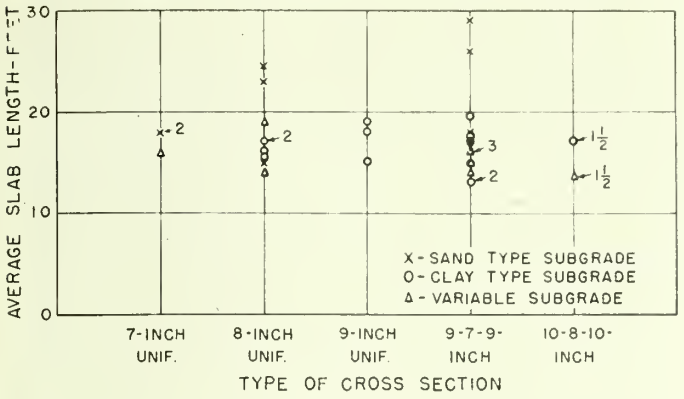


FIGURE 15.—COMPARISON OF AVERAGE SLAB LENGTHS WITH DIFFERENT TYPES OF CROSS SECTIONS EACH POINT REPRESENTS 1 MILE EXCEPT WHERE INDICATED DIFFERENTLY. PAVEMENTS RANGE IN AGE FROM 12 TO 15 YEARS.

concerned, the average daily traffic has had no important effect on the average slab length.

In figure 14 the average slab lengths are plotted against the daily number of wheel loads of different weight classes. Neither the number of wheel loads of a magnitude of 4,000 pounds or greater, nor the number of a magnitude of 6,000 pounds or greater appears to have had any important influence on the amount of transverse cracking that had occurred. Table 5 shows that there were very few wheel loads exceeding 9,000 pounds on these pavements.

While the traffic data used were obtained in surveys made after most of the transverse cracking had taken place, it is thought that the figures do give an indication of the relative amount of traffic that has been moving over the various sections during the earlier life of the pavement.

CROSS SECTION DID NOT INFLUENCE SLAB LENGTH

A study of the relation between the average slab length and the design of cross section is summarized in figure 15. The types of subgrade on which these pavements were laid are indicated by symbols.

To draw definite conclusions from data of the kind presented in figure 15, it is necessary that all other factors are constant throughout. This obviously is not the case. The comparison has other weaknesses. Only the 8-inch uniform thickness and the 9–7–9 inch thickened-edge pavements are represented by a sufficient number of miles to make the values dependable. Also the range of types is not as wide as is desirable. It can only be concluded that the data presented indicate that the shape and thickness of the pavement cross section has little or no influence on the average slab length.

AVERAGE SLAB LENGTHS OF PLAIN AND REINFORCED PAVEMENTS COMPARED

A comparison is made between the average slab lengths of plain and reinforced pavements in figure 16. The reinforced pavements available for this comparison ranged in age from 5 to 7 years and had an average age of 6⅓ years. The average slab lengths of the plain concrete pavements were taken from figures 10 to 11 at the corresponding age of 6⅓ years.

The first group of pavements in figure 16 are plain and partly reinforced. The majority of these pave-

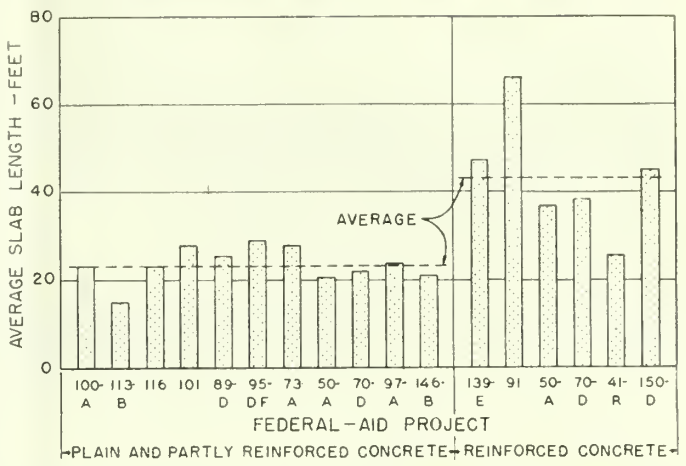


FIGURE 16.—COMPARISON OF THE AVERAGE SLAB LENGTHS ON PLAIN AND REINFORCED CONCRETE PAVEMENTS. THE REINFORCED PAVEMENTS VARY IN AGE FROM 5 TO 7 YEARS AND HAVE AN AVERAGE AGE OF 6.3 YEARS. THE AVERAGE SLAB LENGTHS OF THE PLAIN CONCRETE PAVEMENTS WERE TAKEN AT A CORRESPONDING AGE. THE REINFORCED PAVEMENTS WERE REINFORCED WITH A 60-POUND WIRE FABRIC.

ments had ¾-inch diameter edge bars and projects 100–A, 101, 50–A, and 70–D had small amounts of special reinforcing in certain areas. The details of the special reinforcement are given in tables 1 and 2. A study of the parts of the pavements with special reinforcement indicated that it had no appreciable effect on the structural properties or condition of the pavement. This was probably due to the small amounts of reinforcement used or to the manner in which it was placed.

Project 50–A is the only one in which special reinforcement was placed in a large part of the total length of the pavement. The average slab length on this project was only slightly greater than the average for all the pavements on a similar type of subgrade.

None of the pavements containing special reinforcement, mentioned above, was reinforced with wire fabric. Each of the pavements included in the second group of figure 16 was reinforced throughout with a 60-pound fabric. The percentages of clay and sandy types of subgrades were approximately equal under both the plain and the reinforced pavements. Thus the effect of the type of subgrade may be disregarded in the interpretation of this figure. In some cases the reinforced pavements were single lanes placed along the edges of the old pavement.

It is apparent from figure 16 that the average slab lengths for the pavements containing distributed reinforcement were much greater than were those of the plain concrete pavements at the same age. The average slab length for all the reinforced pavements was more than 80 percent greater than the average for all the plain concrete pavements.

Two projects (50–A and 70–D), represented in this figure, had both plain and reinforced pavements. The type of subgrade under the plain and reinforced pavements on the two sections was the same, yet the average slab lengths for the reinforced pavement greatly exceeded those for the plain pavements.

A 60-pound welded wire fabric, such as was used in the reinforced pavements, is relatively light reinforce-

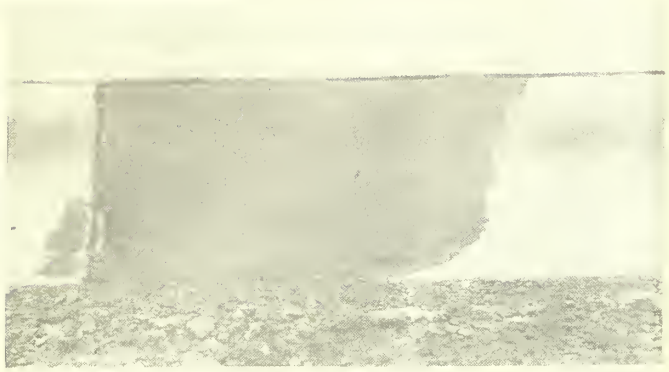


FIGURE 17.—BADLY FAULTED JOINT WITH LEADING SLAB DEPRESSED.

ment for heavy pavement sections. This weight of reinforcement is ordinarily associated with design slab lengths of 50 feet or less. On all but one of these six projects it was used with a constructed slab length of about 100 feet. While it is possible that cracks were present and were held so tightly closed by the reinforcement that they passed undetected, it is believed that such cracks were not numerous.

It is generally conceded that small amounts of reinforcement placed near the neutral axis of a concrete pavement have little or no influence in preventing cracking from wheel loads. The greatest part of the cracking in these pavements apparently had been caused by combined load and warping stresses, and it is possible that where warping stresses are involved small amounts of reinforcement may act in some manner to delay or reduce cracking. Regardless of whether or not this is the explanation, it is indicated that over the present life of these pavements the reinforcement has had the practical effect of increasing appreciably the average slab length and that this effect is likely to continue.

There is another factor that should not be overlooked, however, even though it may not be possible to evaluate its influence. Important changes have taken place in the character of traffic, particularly in that fraction which includes the heavier wheel loads, during the past 10 or 15 years. Speeds have increased greatly and the type of tire equipment has changed radically. For a given wheel load, the strains produced in the pavement slab are probably less now than they were 15 years ago and pavements laid at that time may have been damaged more by heavy wheel loads than were similar pavements placed at a later date. The matter is complex and is only mentioned here to record the fact that this trend exists and is recognized.

ONLY SMALL NUMBER OF CORNER CRACKS FOUND

Corner breaks.—The corner breaks recorded in table 5 are those that were found at expansion joints. The majority of these breaks were located during earlier surveys and are no longer visible because of repairs. At the time of the present survey practically no corner breaks were visible and those that were found were usually at inside corners, next to the longitudinal joints and much smaller than the cracks resulting from load failure sometimes found at outside corners.

Table 5 shows that the number of breaks occurring in the pavements surveyed was very small. Several factors are believed to have contributed to this condition. The concrete pavements, with two exceptions, had edge thicknesses of 8 to 10 inches. The cross sections were of 9–7–9 or 10–8–10 inch thickened-edge design or of 8-, 9-, or 10-inch uniform thickness. Thus, the exterior corners of most of the pavements were relatively heavy. Wide pavements and high operating speeds tend to remove the point of load applications from the extreme edge of the pavement, while better cushioning in the tire equipment tends further to protect slab corners by reducing the impact forces developed at transverse joints.

Discussion of faulting at joints.—Faulting at joints was discussed in a general way for the individual projects in connection with figure 4. The summary of the faulting conditions at joints given in table 5 shows the percentage of joints per lane at which each of three different degrees of vertical displacement were found, the differences indicated in this table being based on the maximum faulting found by measurement in either direction and at any point across the width of the one lane.

It will be noted from these data that on 5 of the older and on 1 of the newer pavements 40 percent or more of the joints had faulted one-fourth inch or more at some point along the joint. The term "newer pavements" includes those built since 1930. On 10 of the older pavements and on 5 of the newer, this same proportion of joints, 40 percent or more, had faulted one-eighth or three-sixteenth inch. Only 1 of the new pavements, project 73–A, had one-sixteenth-inch faults at 40 percent or more of the joints. On 11 of the older pavements and 3 of the newer pavements, 25 percent or more of the joints had faults of one-fourth of an inch or greater. This same proportion of joints (25 percent or more) had faulted one-eighth or three-sixteenth inch on 14 of the older pavements and on 6 of the newer. Only 4 of the pavements had 25 percent or more of the joints with faults of one-sixteenth inch. Two of these were old pavements and 2 were new.

Generally speaking, the most undesirable conditions of faulting were on those projects with the highest percentage of joints that had faulted one-fourth of an inch or more.

Faulting measurements of less than one-eighth inch approach the limit of accuracy of the method used. For this reason some of the joints recorded as having differences of elevation of less than one-eighth inch may not actually be faulted. Because of the large number of measurements made, however, the tendency would be for these errors to cancel out.

It is interesting to compare the condition of the joints in the older and newer sections of projects 73–A and 50–A. On both projects the subgrade material is of the same type over a large part of the length. On project 73–A, a 6- to 10-inch sand and gravel subbase underlies a considerable part of the old pavement and at the time of the survey was in a dry, well-drained condition. The old pavements on both sections were of approximately the same age and had been subjected to approximately the same amount of traffic.

While there was an undesirable amount of faulting in both pavements, the number of badly faulted joints was less on project 73–A than on 50–A, and comparison indicates that the sand and gravel subbase had been of some benefit in reducing faulting at the joints.

The newer pavement of project 73–A was 3 years old at the time of the survey, and that of project 50–A was 5 years old. Provision for load transfer had been in-

cluded in project 73–A, while no load-transfer devices were used in project 50–A. Very little faulting had occurred at the joints of the new pavement in project 73–A, while the faulting at the joints of the new pavement on project 50–A was worse than that of almost any other project studied. For example, table 5 shows that only 2 percent of the joints in the new pavement of project 73–A were faulted one-fourth of an inch or more, while 48 percent of the joints in the new pavement of project 50–A were displaced this amount. This comparison shows very clearly the beneficial action of adequate load-transfer devices in maintaining slab surface alinement at expansion joints.

FAULTING WAS OF TWO TYPES

The data collected in this investigation indicate that there are two distinct types or phases of the phenomenon of faulting. The first type is that in which transverse cracks develop in the pavement near the joints. If the pavement is not reinforced, this crack creates a short slab supported by the subgrade but with no direct structural connection with the other parts of the pavement. The short slabs gradually become displaced vertically at the joints. The magnitude of the displacement is generally less than one-half inch and the joint may be faulted in either direction, that is the slab beyond the joint in the direction of traffic may be either above or below the approach slab and lateral tilting may accompany the displacement. This type of faulting was found to be very general on practically all of the pavements investigated and was present on all of the different types of subgrades.

In the second type of faulting, the slab beyond the joint in the direction of traffic is depressed below the approach slab and the magnitude of the displacement may be as much as one-half inch and frequently exceeds this amount.

A typical example of this type of faulting is shown in figure 17. When such a depression develops, it is frequently necessary to level up the surface with bituminous material for a short distance on the depressed slab.

Table 6 shows the percentage of joints on each project at which a displacement of one-half inch or more had developed. The different miles surveyed in the various projects are separated according to the type of subgrade to indicate the influence of the subgrade on this type of faulting. In the fifth column of this table the percentage of joints at which the slab beyond the joint is depressed is shown, while the sixth column gives the percentage of joints at which the approach slab is depressed. A joint was recorded as faulted one-half inch or more if displacements of this magnitude were measured at any point along the joint. The length of a joint was considered as being the width of one lane.

The table shows only one pavement in which an appreciable number of the joints have the approach slab depressed one-half inch or more. It appears to be a general characteristic that where severe faulting develops at a joint, the slab beyond the joint is depressed with respect to the approach slab.

Table 6 presents data on 120 lane-miles of highway. Of this amount 48.5 lane-miles were on a clay type subgrade, 36 lane-miles were on a sand type subgrade, and 35.5 lane-miles were on variable subgrade. The clay type subgrades were generally A–6 soils and the sand type subgrades were generally A–3 soils. The variable subgrades generally included both the clay and sand types, changing within the mile surveyed.

TABLE 6.—*Data on badly faulted joints*

Federal-aid project No.	Year built	Mile surveyed	General type of subgrade	Joints faulted ½ inch or more	
				Slab beyond joint down	Approach slab down
				Percent	*Percent*
100–A	1925	1, 2, 3	Clay	6	1
113–B	1926	1, 2, 3	do	8	0
116	1924	1, 2	Sand	2	1
		3	Variable	2	1
101	1924	1	do	12	1
		2, 3	Sand	5	0
139–E	1931	1, 2	do	0	0
91	1923	1	do	(1)	(1)
91	1931	1	do	0	0
89–D	1924	1	do	1	0
		2	Clay	1	0
		3	Variable	0	0
95–DF	1926	1	do	3	0
		2, 3	Sand	1	0
73–A	1925	First 1½	Clay	2	0
		Second 1½	Variable	2	0
73–A	1935	First 1½	Clay	0	0
		Second 1½	Variable	0	0
50–A	1923	1, 2, 3	Clay	9	5
50–A	1933	1, 2, 3	do	10	1
70–D	1925	1, 2, 3	do	2	0
70–D	1932	1, 2, 3	do	1	0
131–C	1930	1	Sand	0	0
		2	Clay	5	0
		3	Variable	1	2
97–A	1924	1, 2	Sand	4	0
		3	Variable	3	0
97–A	1928	1, 2	Sand	10	1
		3	Variable	6	0
41–R	1931	1, 3	do	3	0
		2	Clay	9	1
150–D	1932	First 1½	do	4	0
		Second 1½	Variable	1	0
146–B	1925	1, 2, 3	do	4	1
187–E	1918	1	Sand	3	2

[1] No joints.

Considering those projects where 8 percent or more of the joints were faulted one-half inch or more, with the slab beyond the joint depressed, it is found that 72 percent of the lane-miles were on the clay type subgrades, 18 percent were on the sand type subgrades, and 10 percent were on variable subgrades.

Considering those projects in which 5 percent or more of the joints were faulted one-half inch or more, with the slab beyond the joint depressed, it is found that 65 percent of the lane-miles were on clay type subgrades, 23 percent were on sand type subgrades, and 12 percent were on variable subgrades.

Of those pavements of table 6 that were built before 1931, 31 lane-miles were on a clay type subgrade, 26 lane-miles on a sand type subgrade, and 23 lane-miles were on variable subgrade.

In this group 8 percent or more of the joints were faulted one-half inch or more with the slab beyond the joint depressed and of this fraction 62 percent were on the clay type subgrades, 24 percent were on the sand type subgrades, and 14 percent were on variable subgrades.

It is indicated that faulting in which the slab beyond the joint is depressed one-half inch or more occurs more frequently on pavements supported by subgrades of the clay type but that it may also develop in those placed on sandy material.

This type of faulting may develop at an early age. A relatively large percentage of the joints were faulted on the new pavement of project 50–A and on the second mile of project 41–R.

TABLE 7.—*Data on cracking near joints*

Federal-aid project No.	Year built	Lane-miles of pavement	Number of expansion joints per lane-mile	Number of cracks expressed as a percentage of the number of joints within—		Percentage of joints having cracks in slab beyond the joint within—		Percentage of joints having cracks in approach slab within—	
				8 feet of joint	11 feet of joint	8 feet of joint	11 feet of joint	8 feet of joint	11 feet of joint
				Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
100–A	1925	6	56	11	27	4	12	4	14
113–B	1926	6	54	26	65	10	26	14	32
116	1924	6	106	7	21	2	11	3	12
101	1924	6	49	12	37	6	17	4	19
139–E	1931	8	58	0	2	0	0	0	0
91	1931	2	58	2	3	1	2	0	1
89D	1924	6	60	5	20	2	11	2	11
95–DF	1926	6	59	12	36	3	15	7	21
73–A	1925	6	63	22	44	15	30	6	21
	1935	3	105	1	2	0	1	1	1
50–A	1923	6	40	12	17	8	12	2	5
	1933	6	54	11	18	8	12	5	7
70–D	1925	6	30	17	27	9	18	5	12
	1932	3	58	2	3	1	3	0	1
131–C	1930	6	56	2	5	1	3	1	3
97–A	1924	6	58	16	40	7	21	7	18
	1928	6	53	9	23	8	14	2	13
41–R	1931	12	108	1	2	0	1	0	0
150–D	1932	6	53	0	2	0	0	0	0
146–B	1925	6	59	15	30	7	17	5	16
187–E	1918	2	73	5	15	6	8	1	4

FAULTING AT TRANSVERSE CRACKS

Table 5 gives data on differences in slab elevation at both joints and cracks. Faulting measurements were made at only one selected crack in approximately each 100 feet surveyed. This, generally, represented between 15 and 20 percent of the total number of transverse cracks in the pavement. The older and generally the more open cracks were selected for faulting measurements and it is estimated that they were representative of approximately 50 percent of the total number of cracks in the different pavements.

The data on faulting at cracks of table 5 are based on the number of cracks at which faulting measurements were made. These percentages refer, therefore, to the older cracks, constituting about 50 percent of the total, which were probably those at which most of the faulting had developed. It is obvious that the percentage values would be much lower if they were based on the total number of cracks.

It is indicated by the data of this table that the tendency toward faulting at transverse cracks is much less than it is at expansion joints without provision for load transfer. There was only one pavement in which cracks with faults of one-fourth inch or more exceeded 10 percent. This was project 187–E built in 1918.

It is thought that use of longitudinal joints and of edge bars have been beneficial in reducing faulting at cracks. It is believed that the greater faulting at cracks observed on State project 187–E is partly accounted for by the absence of these features.

There is too much difference in ages between the plain and the reinforced pavements to permit a direct comparison of the amount of the faulting which had developed at the cracks. On several of the reinforced pavements the cracks were closed so tightly that faulting measurements were not attempted. These projects are indicated by the absence of entries in table 5. On some of the reinforced pavements a small number of cracks were open slightly and at some of these a certain amount of faulting had occurred.

With few exceptions, on all of the reinforced pavements examined, the slab surfaces adjacent to transverse cracks were in good alinement and in good general condition.

SIGNIFICANCE OF TRANSVERSE CRACKS NEAR EXPANSION JOINTS DISCUSSED

In the course of study of the effect on pavement condition of the omission load-transfer devices at expansion joints, it was noted frequently that transverse cracks had formed in rather close proximity to expansion joints. Table 5 shows the number of such cracks found in the sections surveyed, arbitrarily grouped within zones having a width of 8 feet or 11 feet on either side of the transverse joint.

Table 7 presents the data in a somewhat different manner. The number of cracks is related to the number of joints and also the cracks are divided into those appearing in the approach slab as compared with those in the slab beyond the joint. In each case the number is expressed as a percentage of the number of joints.

Table 7 shows that on five projects the number of cracks within 8 feet of the joints was 15 percent or more of the number of joints. These are projects 113–B, 73–A, 70–D, 97–A, and 146–B. Three of these projects had subgrades that were predominantly clay, one had a subgrade that was predominantly of a sand type, while the subgrade on the fifth was variable.

On six projects the number of cracks within 11 feet of the joints exceeded 30 percent of the number of joints. These are projects 113–B, 101, 95–DF, 73–A, and 146–B. Two of these projects had subgrades that were predominantly clay, three had subgrades that were predominantly sand, while the sixth had a variable subgrade material.

A study of the table shows that there was no consistent difference between the number of cracks near the joints in the slab beyond the joint and the number in the approach slab. The average number of cracks for all pavements having cracks in the slab beyond the joint within 8 feet of the joint was 4 percent of the number of joints, while the corresponding number with cracks in the approach slab was 3 percent. The average number of cracks for all of the pavements having cracks in the slab beyond the joint within 11 feet of the joint was 10 percent, while the corresponding number having cracks in the approach slab was 10 percent.

In the discussion of faulting at joints two types were described. The more serious type was that in which the slab beyond the joint in the direction of traffic was depressed one-half inch or more below the approach slab. The percentage of joints at which this type of faulting had taken place is indicated in table 6 and has already been discussed.

It is desirable to reexamine the data to determine whether there is evidence that impact produced by heavy wheel loads passing over the faulted joints caused transverse cracks to occur beyond the joint.

As stated previously, there are indications, at some of the joints examined, that the faults existing at the time of the survey developed after a transverse crack formed near the joint or that the magnitude of the displacement changed considerably after the formation of a crack. If this were true, a direct comparison of the number of cracks near joints in the slab beyond the joint that are now badly faulted with the number of cracks near joints that are not would not necessarily give a reliable indication of the effect of vehicle impact. A different approach is necessary.

Only those pavements built before 1927 were considered and these were divided into two groups on the basis of the number of badly faulted joints found.

TABLE 8.—*Longitudinal cracking*

Federal-aid project No.	Longitudinal cracking at distances greater than 2 feet from edge of pavement
	Lineal feet per lane-mile
100–A	64
113–B	68
116	160
101	194
139–E	5
91, old	103
91, new	4
89–D	4
95–D	174
73–A, old	85
73–A, new	0
50–A, old	13
50–A, new	2
70–D, old	41
70–D, new	0
131–C	15
97–A, old	79
97–A, new	61
41–R	0
150–D, new	0
146–B	144
187–E	(1)

[1] Practically the entire length.

The conditions of cracking in the vicinity of the joints in one group were then compared with the conditions existing on the other.

The first group includes projects 100–A, 113–B, 101, and 50–A (old section). These were selected from table 6 as having the greatest number of badly faulted joints since at 5 percent or more of the joints the slab beyond the joint was depressed one-half inch or more. Projects 116, 89–D, 95–DF, 73–A (old section), and 70–D (old section) were selected as having the smallest number of badly faulted joints. At only 2 percent or less of the joints on these projects was the slab beyond the joint depressed one-half inch or more.

For a comparison to be valid, it is necessary to assume that the relation between the two groups as to faulting at the joints had not changed appreciably for some time. This is believed to be a reasonable assumption, since in general the pavements that had a large number of badly faulted joints were those on subgrades which are conducive to this condition, while those that had relatively few badly faulted joints were those on more stable subgrades.

In the group of pavements with the larger percentage of badly faulted joints an average of 7 percent of the slabs beyond the joints had cracks within 8 feet of the joints as compared to 6 percent in the approach slabs. In this same group of pavements, 17 percent of the joints were associated with cracks in the slab beyond the joint within 11 feet of the joint as compared to 18 percent for the approach slabs.

In the group of projects with the smaller percentage of badly faulted joints, an average of 6 percent of the joints were associated with cracks in the slab beyond the joint within 8 feet of the joints as compared to 4 percent for the approach slabs. In this group 16 percent of the slabs beyond the joints had cracks within 11 feet of the joints compared to 15 percent of the approach slabs.

ONLY LIMITED AMOUNT OF LONGITUDINAL CRACKING FOUND

Considering only the slabs beyond the joints it was found for the first group of pavements, 7 percent of the slabs had cracks within 8 feet of the joints, while in the second group 6 percent of the slabs had cracks within 8 feet of the joints. Seventeen percent of the slabs in the first group had cracks within 11 feet of the joints, as compared to 16 percent in the second group.

This comparison indicates that regardless of the severity of the faulting, there was as great a tendency for cracks to form in the approach slab as there was in the slab beyond the joint and further that there was as great a percentage of cracks near the joints in the group of pavements that had the fewest badly faulted joints as there was in the group that had the most badly faulted joints.

TABLE 9.—*General condition of expansion joints* [1]

Federal-aid project No.	Year built	Original joint opening	Average joint spacing	Reinforcement [2]	Present joint openings				Remarks
					Closed, open ⅛ inch or less	Open ⅛ to ¼ inch	Open ⅜ to ½ inch	Open full width	
		Inches	*Feet*		*Percent*	*Percent*	*Percent*	*Percent*	
100–A	1925	1	94	Special	56	31	13	0	Joints filled near pavement edges with a compressed soil.
113–B	1926	1	98	None	37	42	21	0	Do.
116	1924	¾	50	do	60	25	15	0	Do.
101	1924	¾	108	Special	0	0	100	0	Do.
139–E	1931	1	91	Wire fabric	0	0	100	0	Joints filled near edges with a compressed soil.
91	1923			None					No expansion joints.
91	1931	1	91	Wire fabric	0	0	100	0	Joints filled near pavement edges with a compressed soil.
89–D	1924	¾	88	None	67	22	11	0	Do.
95–DF	1926	1	90	do	40	35	25	0	Do.
73–A	1925	1	84	do	50	30	20	0	Do.
73–A	1935	1	50	Wire fabric	0	0	0	100	Joints free.
50–A	1923		132	Special	100	0	0		Some joints are open slightly, but these were filled with a very hard formation.
50–A	1933	1	98	Wire fabric	0	0	0	100	Joints free.
70–D	1925	1	176	Special	53	40	7	0	Joints filled near pavement edges with a compressed soil.
70–D	1932	1	91	Wire fabric	0	0	0	100	Joints still free, but some sand had entered.
131–C	1930	1	94	Fabric in part	0	32	36	32	Joints filled near pavement edges with a compressed soil.
97–A	1924	¾	91	None	100	0	0	0	Some joints were open slightly, but these were filled with a compressed soil.
97–A	1928	1	100	Fabric in part	0	73	27	0	Joints filled near pavement edges with a compressed soil.
41–R	1931	¾	49	Wire fabric	0	0	0	100	Do.
150–D	1932	1	100	do	0	0	0	100	Some of the joints were free, while others were filled near the pavement edges with compressed soil.
146–B	1925	1	90	None	68	21	11	0	Joints filled near pavement edges with a compressed soil.
187–E	1918		72	do	0	0	100		Do.

[1] These observations were made during the summer
[2] For details see tables 1 and 2.

FIGURE 18.—A SECTION OF PAVEMENT TYPICAL OF THOSE STUDIED.

FIGURE 19.—SCALING IN CORNERS FORMED BY EXPANSION JOINT.

There is no evidence in these data that the impact at badly faulted joints was responsible for the cracking that had developed near the joints. In this connection it is to be noted that the number of cracks found near joints appears to bear no direct relation to the amount of heavy traffic that had been on the pavement.

It is believed that the recent researches concerning the structural action of concrete pavement slabs (*5, 6, 7, 8, 9*) offer an explanation of the manner in which the type of transverse cracking that is being discussed occurs.

Published reports (*5, 6, 7, 8, 9*) show that the critical stresses in concrete pavement slabs are combinations of tensile stresses from wheel loads with tensile stress already present because of restrained warping. It will be found, by a study of the distribution of the stresses caused by restrained warping that the magnitude of the important longitudinal warping stresses is relatively low near the joint edge but increases to approximately a maximum at a distance of 8 to 10 feet from the slab end. These maximum longitudinal warping stresses cannot combine with the maximum stress caused by impact of a wheel load at a faulted joint because the two do not coincide in position. It is suggested by the warping data however that the longitudinal warping stresses combined with the stresses produced by heavy wheel loads applied 8 or 10 feet from the joint may be the cause of the transverse cracks that so frequently develop at this distance from the slab ends.

The longitudinal cracking in the pavements studied apparently could be divided into two types. Cracks of the first type had developed near the edges of the pavement and did not appear to have been caused by any stress condition that should normally be present in concrete pavements. This type of defect is discussed later under the heading "infiltration cracks." Cracks of the second type were found at or near the longitudinal center line of the pavement lanes and appear to have been caused by stresses that are to be expected in concrete pavements.

The amount of this type of cracking per mile per lane of pavement for the different projects is shown in table 8. In preparing this table a crack was considered as being longitudinal if its direction did not depart more than 45° from the longitudinal axis of the pavement. All of the projects studied except 187–E had a longitudinal center joint.

The table shows that the only project with a serious amount of longitudinal cracking was 187–E. This is the 16-foot pavement laid in 1918 without a longitudinal center joint. There were 5 other pavements in which the amount of longitudinal cracking per mile exceeded 100 feet, or 2 percent of the total length. These were all old pavements built between 1923 and 1926 inclusive. The longitudinal cracking in these pavements consisted generally of short longitudinal cracks between two transverse cracks or between a transverse joint and a transverse crack. Others appeared to start from a joint or crack and extended a short distance in a direction approximately perpendicular to the joint or crack. There were other longitudinal cracks that appeared to have been caused by frost action.

There was not enough longitudinal cracking to justify definite conclusions regarding the influence of the type of subgrade or the type of pavement cross

section. It was noted, however, that a greater amount of longitudinal cracking had occurred in the pavement laid on sandy type subgrades. The reason for this is not known.

GENERAL CONDITION OF THE JOINTS DISCUSSED

Table 9 is a general summary of the condition of the joints in the various sections of pavements surveyed. The data are based on a detailed examination of five to eight typical joints per mile of pavement, or approximately 12 percent of the total number of joints in the pavement. As a part of this examination the shoulder material was dug away from the edge of the pavement at the joint for the full depth of the pavement. A surface examination was made of many other joints and they were found to be in approximately the same condition as those examined in detail.

Measurements of the widths of the joint openings were made during the hot weather of midsummer when the pavement was expanded. The temperature of the pavement at the time these measurements were made had little effect on joint openings in the plain concrete pavements because they had cracked into relatively short slab lengths. In the reinforced pavements, however, the widths of the joint openings were more responsive to changes in the temperature of the pavement and this fact should be considered in interpreting the data of the table.

On two pavements 100 percent of the joints were classified as closed and on eight pavements 50 percent or more were similarly classified. These pavements were built before 1926 and showed many transverse cracks. Five of the eight were not reinforced while the remainder had special reinforcement in certain parts of the pavements.

There were four projects (101, 139–E, 91, and 187–E) where 100 percent of the joints were open between three-eighths and one-half of an inch. Of the part of project 101 that was surveyed, approximately one-fourth was reinforced with special reinforcement (see table 2), and of the remaining three-fourths, a large part contained ¾-inch edge bars running continuously through the joints. It is possible that there was sufficient bond between the edge bars and the concrete to prevent free slippage and that the bars had a tendency to hold the cracks closed, and the joints partly open. Project 187–E was plain concrete pavement built in 1918. It was, therefore, surprising to find the joints open as much as they were. The explanation may lie in the relatively small amount of transverse cracking that had developed in this pavement.

There were five pavements, in which all of the joints were found to be open a normal amount for the temperature at the time of the survey. These were all relatively new pavements reinforced with a 60-pound wire fabric.

It is indicated by the data in table 9 that the most important cause of the closing of the expansion joints on the plain concrete pavements studied was the formation and the subsequent opening of transverse cracks. Permanent growth of the concrete may have been present but it is not possible to determine its extent from these data.

As indicated in the last column of this table, the openings that remained at the expansion joints were usually filled near each pavement edge with tightly compressed soil. This condition appeared to be worst in pavements having granular material on the shoulders and the density of the foreign matter seemed to be greatest near the edges of the pavement slabs. The foreign material in the joints of old, badly cracked pavements was particularly dense.

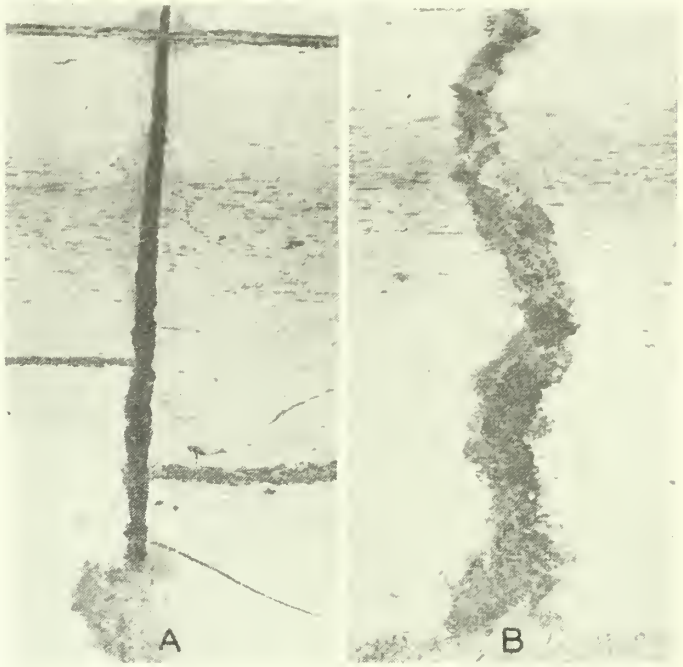


FIGURE 20.—A, NORMAL JOINT IN A TYPICAL OLD PAVEMENT; B, NORMAL CRACK IN A TYPICAL OLD PAVEMENT.

FIGURE 21.—A BADLY SCALED AND DISINTEGRATED PAVEMENT.

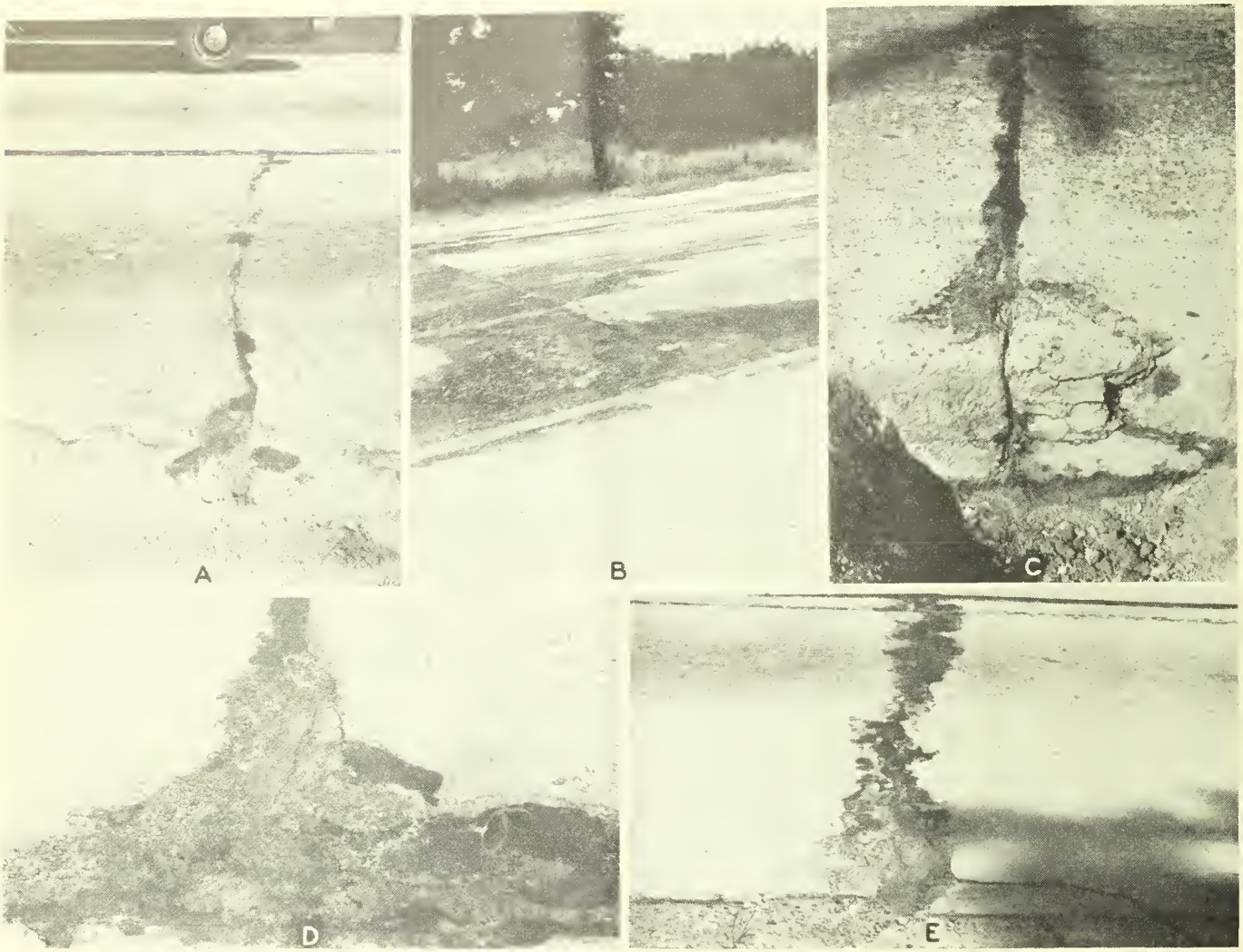


FIGURE 22.—A, SURFACE DISINTEGRATION AT CORNERS FORMED BY CRACK; B. SCALING AT CORNERS FORMED BY JOINT, COVERED WITH CRACK FILLER; C, SMALL CORNER FAILURE AT CLOSED EXPANSION JOINT; D, SCALING AT CORNERS FORMED BY CRACKS COVERED WITH CRACK FILLER; AND E, SMALL CORNER FAILURE AT CORNER FORMED BY CRACK.

The more recently built pavements in Michigan have been reinforced and the joints have been filled with an impregnated fiber joint filler. There is indication that these changes will improve the functioning of the expansion joints.

On a number of the projects the gravelly shoulders were slightly higher than the pavement. Some of the shoulder material found its way on to the pavement and eventually into the joints. Lowering of the shoulders would possibly help to prevent infiltration.

The conclusion to be drawn from this study is that if the normal widths of the expansion joints are to be maintained it will be necessary either to eliminate cracking or to place sufficient reinforcement to hold the cracks closed. It will be necessary also to prevent foreign material from entering the joints, particularly near the edges of the pavement.

There is one important observation in this connection. In spite of the many tightly closed expansion joints there appear to have been very few blow-ups in the pavements. Apparently the open cracks must act in some degree as expansion joints relieving the worst of the compression caused by expansion.

SPALLING, SCALING, AND OTHER DAMAGE AT JOINTS AND CRACKS

A pavement typical of those studied is shown in figure 18. The inside lanes of this highway were approximately 13 years old and the outside lanes were approximately 4 years old.

The dark areas on the pavement in this and other photographs, are as a general rule areas where the surface had scaled or disintegrated and repairs have been made with bituminous mixtures. This illustration shows that the damage to the old pavement is most evident in the vicinity of the corners formed by the joints and cracks. A close-up view of this condition is shown in figure 19. This is representative of one of the more badly scaled joints in a pavement in average condition for an age of 13 years.

Close-up views of a joint and a crack typical of those on one of the older pavements examined are shown in figure 20. This surface was in better than average condition for pavements of the older group. Some spalling can be noted along the edges of both the joint and the crack. It is typical that the extent of the spalling was greater at the crack than at the joint.

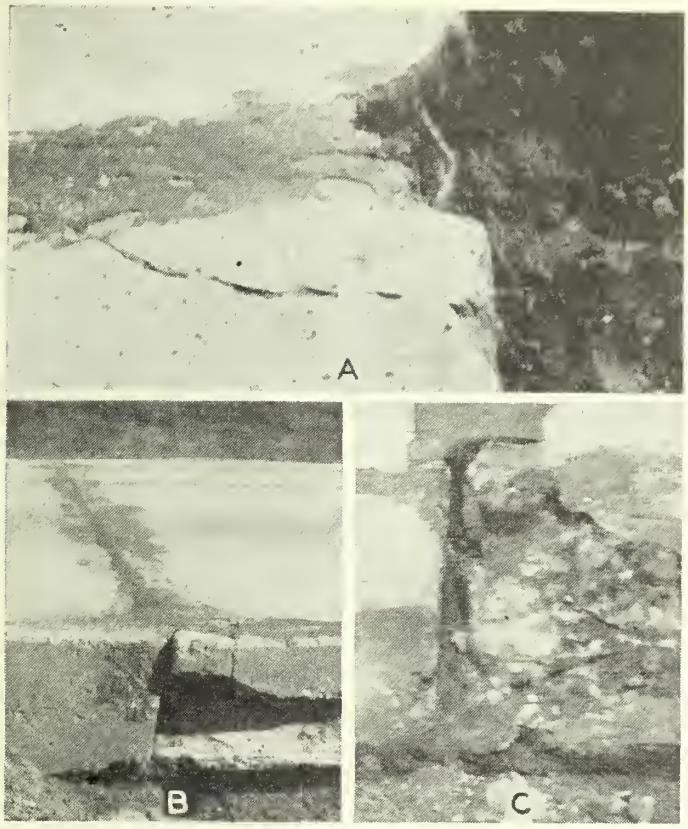


FIGURE 23.—A, BUCKLED EDGE BAR OF A SMALL CORNER FAILURE; B, SMALL CORNER FAILURE AT A JOINT; C, SMALL CORNER FAILURE ON RELATIVELY NEW REINFORCED PAVEMENT.

A general view of one of the more badly scaled and disintegrated pavements is shown in figure 21. The center and right lane of this pavement were approximately 13 years old, while the left lane was relatively new. The new pavement was in good condition. The scaling and disintegration on the old surface was largely in the vicinity of joints and cracks and especially in the corners formed by the joints and cracks. Figures 22, A, and 22, D show in detail the character of the disintegration which has developed at some of the more badly damaged areas at a crack. Figure 22, B shows similar areas at a joint where bad disintegration had developed.

CAUSES OF INFILTRATION CRACKS EXAMINED

On all of the pavements the greatest part of the scaling and disintegration was in the vicinity of joints and cracks, and it apparently starts at and spreads from these points. It was observed also that there was a greater tendency for this type of deterioration to develop at cracks than at joints.

A large number of small corner breaks, of the type shown in figures 22 and 23 were found in a number of the pavements. That in figure 22, C is typical of those found at closed joints in plain concrete pavements. These failures apparently were caused by high compression at the corners, when the pavement was in an expanded condition.

The corner breaks shown in figures 22, E and 23, B are typical of a number found at the joints and cracks in the pavements of projects 116 and 91. In both of these pavements continuous plain round edge bars were used. On project 116 expansion joints were spaced at intervals of approximately 50 feet, while on project 91 there were no expansion joints. At a number of corner breaks on these two pavements, an examination showed that the edge bars had buckled and apparently caused the corners to fracture. A close-up view of one of these buckled bars is shown in Figure 23, A. The edge bars in this pavement were in continuous bond and extended across the expansion joints with no provision for relative movement of the steel with respect to the concrete at these joints.

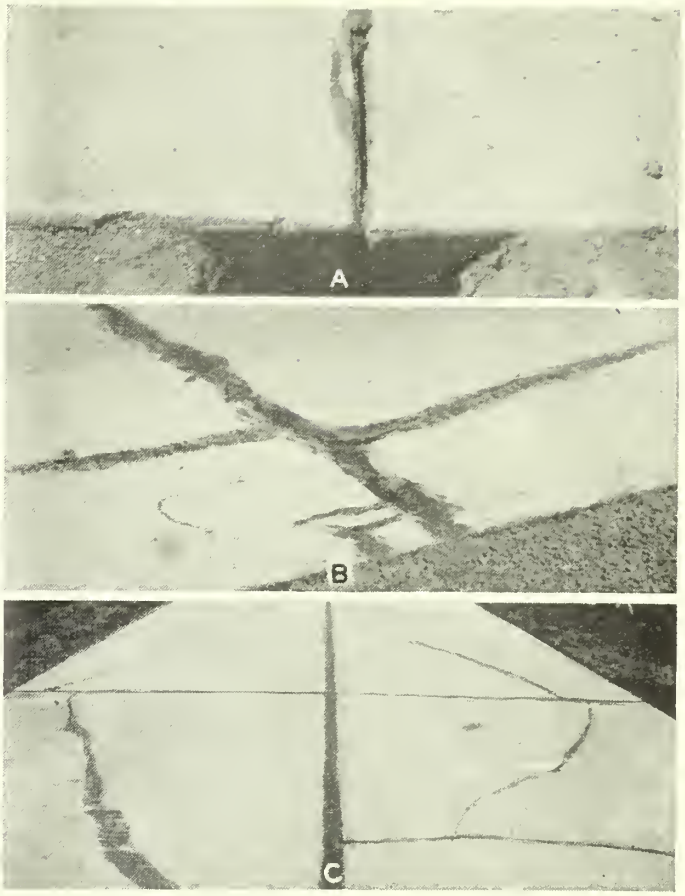


FIGURE 24.—A, INFILTRATION CRACK IN EARLY STAGE OF DEVELOPMENT IN REINFORCED PAVEMENT; B, TYPICAL INFILTRATION CRACK; C, TYPICAL INFILTRATION CRACKS THAT HAVE DEVELOPED TO A SERIOUS DEGREE.

The corner breaks shown in figure 23, C is typical of a large number found in the relatively new reinforced pavement of project 139–E. This joint was open approximately one-half inch but near the edges of the slab it was filled with a tightly compacted sand formation. Apparently breaks of this type were caused by highly localized compression at the corners when the pavement was expanded. The failure indicates the importance of keeping the joints free of foreign matter even in reinforced pavements. It is true that the reinforcement tends to hold the cracks closed and in this way helps to retain the normal width of the joint, but if the cracks are held closed and the joint becomes filled with incompressible foreign material the reinforced pavement may be restrained more than a plain concrete pavement.

"Infiltration crack" is a name that has been applied by engineers of the Michigan State Highway Department to a peculiar type of cracking found in many of the concrete pavements in Michigan. Figure 24 shows photographs of typical infiltration cracks.

FIGURE 25.—INFILTRATION CRACKING ON STATE PROJECT 3924 C–1.

Figure 24, A shows an early stage of development. Figure 24, B shows a more advanced stage while figure 24, C shows a number of these cracks that have developed to a serious extent.

Cracks of this type generally start as a longitudinal crack, but have a tendency to progress toward the longitudinal joint in a curvilinear path. In the pavement examined, it was found that the cracks almost invariably begin at transverse joints at a point near the edge of the pavement, the distance from the edge being generally somewhere between 8 and 36 inches.

The Michigan State Highway Department had made an earlier study of this type of cracking and found that the joints at which these cracks occurred were filled for a short distance near the edges of the pavement with a hard compacted formation of earth that had filtered in from the shoulders. With the outer ends of the joint tightly filled, part of the pavement has room in which to expand while the other part along the pavement edges is restrained. Thus, there may be a tendency for a splitting or shearing action at the joints as the pavement expands.

Among the pavements selected for study of joint conditions, the greatest number of infiltration cracks were found on projects 101, 95–DF, 146–B, 91 (old), and 139–E. All of these, with the exception of 139–E, were plain concrete pavements constructed before 1927. The pavement on project 139–E was a reinforced pavement laid in 1931 and on it infiltration cracks were in a very early stage of development. Two of the pavements had edge bars and three did not. All of the pavements with an appreciable number of these cracks were laid on a sandy subgrade and, as in the earlier Michigan investigations, it was found that the joints at which these cracks occurred, were filled near the edges of the pavement with compressed earth.

Development of these cracks was not serious on any of the pavements included in the formal study. The greatest number was found on project 95–DF. This pavement was built in 1926 and at the time of the survey, there was an average of 13 such cracks per lane-mile. The cracks had an average length of approximately 8 feet and had not created a serious condition.

A special study was made of one pavement on which the condition caused by this type of cracking had become somewhat more serious. This was a State project on route M–89 between Battle Creek and Gull Lake a plain concrete pavement built in 1930. It was located on a sandy subgrade of group A–3. There was no heavy commercial traffic and general traffic was light. Figure 25 show typical examples of the infiltration cracks. Often the cracks were wide and they greatly weakened the pavement. They were found at intervals over the 5 miles of pavement with the exception of short sections that were densely shaded.

It is of interest to examine the data for evidence, as to the correctness of the theory that the impacted material at the joint ends is responsible for the formation of the infiltration cracks.

While it is true that those joints at which these cracks had developed were found to be tightly packed with soil material near the pavement edges, it is also true that this was a more or less common condition in the older pavements and many joints that were filled with earth had not developed these cracks.

Joints containing compressed soil near the pavement edges were found on both clay and the sand subgrades yet the type of cracking being discussed was largely confined to pavements on sandy subgrades. It is possible that the material in the joints of the pavements on clay was not as dense as that in the joints where sandy material obtains but a careful examination in the field of the compacted material of both types failed to show any noticeable difference. It is possible that a difference in compressibility was not revealed by the field examination.

It might be expected that high compression against the ends of two abutting slabs would, if confined to a small area, result in a spalling or shearing failure at the extreme corners. Examination of a great many corners at joints where infiltration cracks had occurred disclosed only a few such failures.

Figure 26 was prepared to study the possible effect of combined load and temperature warping stresses in causing infiltration cracks. The load stresses were obtained by applying a static load at points 1 foot apart across the free end of a pavement slab. These stresses are the averages obtained from tests on several slabs. In each case the center of the loaded area was placed 9 inches from the end of the slab and the critical stress was measured directly under the load in the direction parallel to the joint. The warping stresses were measured at corresponding points also in a direction parallel to the transverse joints at a time when the warping stress conditions were critical.

It is believed that the stresses shown by the curves of this figure are typical of those existing across the ends of slabs having a width of approximately 10 feet

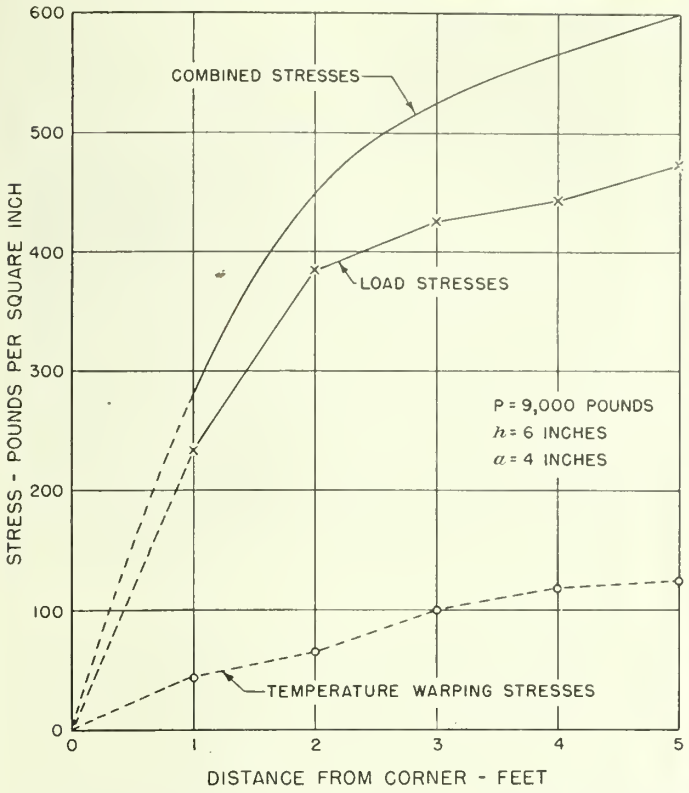


FIGURE 26.—TYPICAL COMBINED LOAD AND WARPING STRESS CURVE ACROSS FREE END OF PAVEMENT SLAB.

and with no provision for load transfer. It is evident that for the conditions described, the load stresses are far more important than the warping stresses.

The combined stresses are relatively low near the edges of the pavement, but increase with the distance from the corner. At distances greater than 2 feet from the edges, the combined stresses are higher than is desirable. Wheel loads of the magnitude used in these tests (9,000 pounds) moving in a normal position can create high combined stresses in that part of the slab end where many of the infiltration cracks begin. From this it might be assumed that combined load and warping stresses are the cause of the cracks, were it not for the fact that some cracks have their origin approximately 8 inches from the edge of the pavement and figure 26 indicates that the combined stresses are definitely low in this region. Also the pavement with the greatest number of these cracks and on which they have progressed to the greatest degree carried only light traffic. It appears that combined load and warping stresses, while they may contribute to the stress condition that causes the rupture, can hardly be the sole cause.

It seems more likely that failure may be due to a complex stress condition produced by a combination of forces derived from wheel loads, warping restraint, and the eccentric expansion against a partially filled expansion joint. The Michigan State Highway Department is attempting to develop a method of sealing joints that will prevent infiltration of material from the shoulders with the idea that this may prevent the development of these cracks.

Until the causes of this type of cracking are definitely established, it may be desirable to place one or two

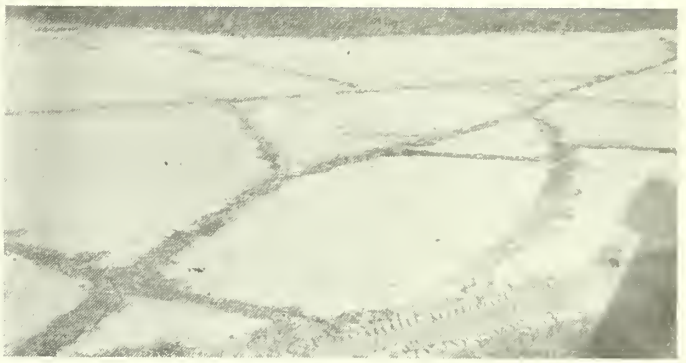


FIGURE 27.—CRACKING CAUSED BY DIFFERENTIAL FROST HEAVING AT INTERIOR OF PAVEMENT.

FIGURE 28.—CRACKING NEAR JOINTS CONTAINING LOAD-TRANSFER DEVICES THAT RESIST UPWARD FLEXURE.

½-inch reinforcing bars across the end of the slab at the expansion joints in pavements placed on sandy subgrades. Such reinforcement would probably keep the cracks closed should they form.

ADDITIONAL PAVEMENTS EXAMINED

In conjunction with this general study, certain pavements not selected for detailed examination were inspected. The observations made are included in this report as general information pertinent to the subject.

The first pavements inspected in this way were located in the Upper Peninsula. They had been built within the preceding 3 or 4 years and all of them contained joint devices that permitted free downward bending but seriously restrained upward bending at the joint.

In the past, trouble with differential frost heaving had been experienced in the subgrades of pavements in the Upper Peninsula. Considerable effort was being made to prepare the subgrades for new pavements in a manner to avoid this trouble, but the steps taken apparently had not been entirely successful.

FIGURE 29.—LONGITUDINAL CRACKING IN PAVEMENT WITH RIGID LONGITUDINAL JOINT.

RIGID JOINT DEVICES MAY CAUSE CRACKING

Figure 27 shows a typical pattern of cracking where differential frost heaving had occurred under a slab in an area away from a joint. The pattern of cracking which developed from the same cause in the vicinity of a joint appeared to be different, depending upon whether the joint had a load-transfer device and varying with the type and rigidity of the device.

When the joints do not have load-transfer devices, the ends of the adjoining slabs are free to move vertically and independently of each other. This frequently results in high or badly faulted joints, but it prevents development of certain stresses when heaving occurs.

Frost heaving beneath free transverse joints, if sufficiently severe, causes simple transverse cracks to form near the joints but apparently does not cause the very irregular diagonal and crisscross fracture that results from severe frost heaving under the slab interiors. On the sections of pavement that were studied intensively a number of cases were noted where frost heave had broken the interior area of a slab in the manner shown in figure 27. Such failures were not observed near the free joints although it was reported that certain of the joints examined were known to have been severely heaved by frost action in previous winters. The slabs at these points were cracked transversely near the joints but no other cracking was observed.

When load-transfer devices of a type that resists joint flexure are used at transverse expansion joints, differential frost heaving causes cracking in the vicinity of the joint similar in pattern to that which develops at the interior of a pavement slab. Two joints that illustrate this action are shown in figure 28. Figure 28, A shows a joint on FAP 265–B, while figure 28, B shows a similar one on FAP 163–F. Both of these were new reinforced pavements and, when inspected, the cracks were being held closed by the reinforcement.

Another type of cracking observed during the general inspection of certain pavements is that shown in figure 29. The longitudinal cracks developed along each side of the longitudinal joint and were continuous over a large part of the total length of the pavement. This type of cracking was investigated in Wisconsin (*2*) and was found to be associated with a soil in which frost heaving occurs and also with close spacing of tie bars across the longitudinal joints. The longitudinal joints in the Wisconsin pavements in which this type of cracking had occurred were of the tongue-and-groove type with tie bars spaced 24 inches. This type of cracking was not found in pavements that were similar with the exception of a tie bar spacing of 48 inches.

The longitudinal joint used in the Michigan pavement in which this type of cracking was observed is of the tongue-and-groove type with tie bars spaced at 20 inches. It is quite possible that the cracking was in some measure caused by the rigidity at the longitudinal joint which resulted from the use of a relatively large amount of tie steel.

In the Arlington investigation of the structural behavior of concrete pavements, (*5*, *6*, *7*, *8*, *9*) it was found that some of the most important stress conditions in pavement slabs of normal thickness and normal size are those in the longitudinal direction of the pavement caused by variations in the temperature and moisture condition of the concrete. Of these two it seems probable that the more important are the stresses caused by restrained warping when a large temperature differential develops in the pavement. At such times the warping stresses are most critical. In certain parts of the slab the tensile stress from restrained warping combines with the tensile stresses caused by loads and the resultant stresses are frequently very high, considerably higher than is desirable as a working stress. The practical implications of these combined stresses have been quite thoroughly discussed in two comparatively recent papers (*1*, *3*).

LOAD TRANSFER AT JOINTS DESIRABLE TO PREVENT FAULTING

The warping stresses increase with an increase in the length or width of the slabs up to a point where the dimension of the slab is sufficient to effect complete restraint. For example, in a long narrow slab the warping stresses are high in the longitudinal direction but low in the transverse direction. In very short narrow slabs the warping stresses are low in both directions. Except in very small slabs, the magnitude of the load stresses is not greatly affected by the size of the slab. The magnitude of the load stress is affected however by the distance between the center of load application and the edges of the slab. For

example, directly under a load applied at the extreme edge of a slab the stress in the direction perpendicular to the edge is zero. This same load applied at a distance of approximately 3 feet from the edge causes a stress approximately equal to that which would be caused if it were applied at the interior of a large slab.

Westergaard has shown by theory that when load stresses alone are responsible for the cracking of a concrete pavement slab the tendency is for the slab to break down into pieces that are approximately 3 to 4 feet square (*10*). Researches of the Public Roads Administration have indicated that normal wheel loads alone should not be expected to cause critical stresses in slabs of 7 inches or more in thickness. It is only when the stress from a wheel load combines with a stress from restrained warping that the combined value reaches a magnitude that may rupture the slab. If the dimensions of the slab are reduced by closer spacing of joints the magnitude of the warping and hence of the combined stresses is reduced. Experience shows that if joints are not provided, pavements tend to break down into relatively small units.

Both the load stresses and warping stresses are affected by the thickness of the pavement. Other conditions being equal, load stress increases as the thickness of the pavement is reduced, while in slabs of appreciable length the warping stress decreases.

When consideration is given to the stresses in concrete pavements resulting from wheel loads and temperature variations, it is reasonable to assume that a large percentage of the transverse cracking in long slabs is caused by critical combinations of load and warping stresses. This appears to explain the presence of the transverse cracks often found near the expansion joints as well as those at greater distances from the joints. Other transverse cracking may be caused by frost action or settlement of the subgrade.

The cause of infiltration cracking is not definitely known, but there is strong evidence that this form of cracking is not caused by the load stresses alone.

The warping stresses are low in the vicinity of free corners of pavement slabs. Therefore, it is reasonable to believe that corner breaks, involving relatively large areas, are caused largely by the load stresses.

There is a present tendency to use for load transfer in the joints, devices designed primarily to give high shearing resistance when loads are applied on one side of the joint. Sometimes these load-transfer devices are designed without regard to the effect they may have on stresses near the joints from causes other than loads. A realization of the importance of other stress conditions is necessary if a joint structure is to be properly designed.

In the report of the Arlington investigation (*8*) it was concluded that the main functions of joints in concrete pavements are to permit the pavement to expand, contract, and warp freely. Load-transfer devices and devices to hold the two sides of a joint in a common plane should be designed to correct certain weaknesses at the joint edges of the pavement and should be designed in a way which will not interfere with the main functions of a joint.

Another function of joints in concrete pavements and one that is not often stressed is to give to the pavement sufficient flexibility for adjustment to changes in the shape of the subgrade. A differential settlement or heaving of the subgrade of only a fraction of an inch will leave a portion of a completely rigid pavement unsupported by the subgrade. Such a condition is especially apt to develop on subgrades subject to large or irregular volumetric changes with freezing or with changes in the moisture content. If the pavement contains a sufficient number of properly designed, sufficiently flexible joints, it is obviously free to adjust itself to the changed condition of the subgrade without serious cracking.

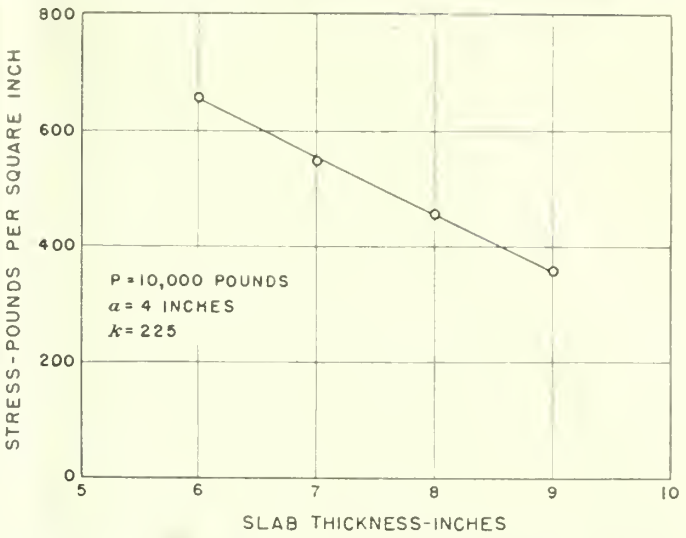


FIGURE 30.—COMBINED LOAD AND WARPING STRESSES MEASURED AT THE EDGES OF FREE TRANSVERSE JOINTS. THE LOAD STRESSES WERE MEASURED DIRECTLY UNDER THE LOAD IN THE DIRECTION PARALLEL TO THE JOINT EDGE. THE WIDTH OF THE SLAB WAS 10 FEET.

The depth of a slab of uniform thickness is determined generally by the stress conditions at the edges or weakest part. For this reason it has been concluded (*3*) that edge strengthening is not necessary at the edges of adequately designed uniform-thickness pavements. The interior depth of a thickened slab should be determined by the stress conditions at the interior or strongest part of the pavement. Therefore, in such a pavement the transverse edges formed by joints and cracks are a source of potential weakness unless provision is made for strengthening them. A load-transfer device is frequently used for this purpose.

Sometimes, because of an arbitrary selection of cross section the interior thickness of a thickened-edge pavement may be greater than is necessary to satisfactorily resist the combined stresses at the interior. In such cases the stresses along transverse joint edges from combined load and temperature effects may not be excessive and stress reduction through edge support may not be necessary. However, this is true only for slabs with relatively great interior thickness.

To study this condition the combined load and warping stresses were measured at the edges of free transverse joints in slabs of four different thicknesses. These stresses are shown in figure 30. The warping stresses were measured under critical temperature conditions. The load stresses were measured directly under a load of 10,000 pounds in the direction parallel to the joint. The subgrade under this pavement was a silty loam of group A–4.

These data show that the combined load and warping stresses, at the transverse edges of a free joint, decrease as the thickness of the pavement is increased. This is because the width of the pavement slabs on which these stresses were measured was only 10 feet, and the warping stresses in the transverse direction of slabs of this

width are not large. The magnitudes of the combined load and warping stresses at the edges of free joints in slabs of different thicknesses, depends, therefore, very largely upon the magnitudes of the load stresses.

Concrete of the class generally used in pavements has a transverse bending strength ranging approximately from 600 to 1,000 pounds per square inch. It is apparent that the stresses shown in figure 30 for the 6- and 7-inch slabs are higher than desirable, but the stresses for the 8- and 9-inch slabs are no higher than the combined stresses which frequently occur at the interior of slabs of normal size and thickness.

The data obtained in the survey show that faulting has occurred at the joints of the majority of the pavements investigated. While the severity of the faulting appears to be influenced to some extent by the type of subgrade material, it was found that faulting had developed to an undesirable degree on all types of subgrades observed. Some faulting was found at cracks, but the severity at cracks was definitely less than at joints.

If the smoothness of the pavement surface at joints is to be maintained, provision must be made to insure the maintenance of the slab alinement and this will be as necessary for pavements of uniform thickness as for those with a thickened edge.

Devices for connecting the ends of abutting slabs at an expansion joint thus divide themselves into two classes: first, those whose function is to maintain surface alinement and reduce the critical edge strains and, second, those whose sole purpose is to maintain the surface alinement.

Load-transfer devices of the first class are the most troublesome since the requirements for a satisfactory design are more severe. The general principles of action of this type of joint have been discussed in some detail in the report on the Arlington research (*8*). Further research is needed to provide additional information concerning the efficacy of some of the designs that have been used or proposed for use.

In the second group, where reduction in critical slab stresses is not a consideration, the problem of designing suitable units is less complex, but even so, there are essential requirements that must be taken into account. For example, the device must provide sufficient strength to resist the forces that tend to produce faulting without introducing serious restraint to slab warping.

SUMMARY AND CONCLUSIONS

The primary object of this study was to determine if possible whether the condition of the pavements indicated a need for load-transfer devices at transverse joints. The method was to examine typical sections of selected pavements for (1) evidence of structural weakness in the vicinity of transverse joints and (2) evidence of other undesirable conditions attributable to the absence of structural connection between the ends of abutting slabs. Certain collateral information has been developed and observations concerning this have been included.

In considering the evidence, it should be kept in mind that the pavements selected for study in general were old pavements on the more heavily traveled routes and the transverse joints had no provision for load transfer. The thicknesses of the interior portion of the pavements selected for detailed examination were 7 inches or more.

Evidence of structural weakness in the vicinity of transverse joints.—Pavement slabs, like other structures, show evidence of structural weakness by cracking under the action of the forces to which they are subjected. Hence, the character of the cracking found in pavements that have been in service for some time, indicates rather clearly whether the pavement is strong enough to resist the forces normally to be expected. In such an examination, it is necessary to distinguish between normal and abnormal conditions of stress.

The indications that bear on the structural weakness in the vicinity of joints are:

Corner breaks.—Failures at slab corners are of two types. One is associated with structural weakness and in failures of this type, a wheel load applied in the corner region breaks off a considerable area of the slab corner in a manner that is characteristic and readily recognized. Failures of this type were almost completely absent in the pavements examined.

The other type of corner failure is not associated with structural weakness. It was rather prevalent and will be mentioned later.

Longitudinal cracks.—Longitudinal cracks are most likely to be caused by combined warping and load stress. If a pavement is divided into articulated lanes approximately 10 feet wide the warping stress component is not large and longitudinal cracking ordinarily does not occur if the pavement is thick enough to keep load stresses within safe values. Longitudinal cracking at a transverse joint may indicate structural weakness from inadequate slab thickness or lack of edge support.

Such cracks were found in a few isolated instances, extending from a transverse joint to a transverse crack, but in general, it may be said that indications of structural weakness, as shown by longitudinal cracking were almost wholly absent from the pavements examined in detail.

Other special forms of longitudinal cracking were observed and will be mentioned later.

Transverse cracking in the vicinity of transverse joints.—Formation of transverse cracks near transverse joints has been studied carefully in the field and has been analyzed from the standpoint of critical combined stresses.

It was found that the number of transverse cracks near the transverse joints is approximately the same on the approach slabs (in the direction of traffic) as it is on the slabs beyond the joints. This condition is about the same on pavements with a large number of badly faulted joints as on pavements with relatively small number of such joints.

The total number of transverse cracks near transverse joints was approximately the same in pavements with a large number of badly faulted joints in as those with a small number of such joints.

The data indicate that the majority of the transverse cracks that occurred near transverse joints were caused by combined load and warping stresses across the section where the crack formed and that the absence of load transfer in the nearby joint was not primarily responsible for their formation.

First conclusion.—From the evidence it must be concluded that in the pavements examined in detail in this survey, there is little evidence of structural weakness in the vicinity of the transverse joints.

UNDESIRABLE CONDITIONS ATTRIBUTED TO ABSENCE OF STRUCTURAL CONNECTION BETWEEN THE ENDS OF SLABS

Dowels and similar devices between the slab ends that come together at a joint are generally considered to have two main functions: To enable one slab end to help support the other as a load passes and to maintain the alinement of the pavement surface in the vicinity of the joint. The importance of the first function depends upon the slab design and it has already been discussed. The second function is important as it affects the smoothness, the appearance, and the structural durability of the pavement.

The faulting of joints, that is, the vertical displacement of one slab end with respect to the other, was found to be prevalent to an undesirable extent in all the older pavements examined. While it was evident in pavements on all of the types of subgrades, pavements laid on clay-type subgrade materials were found to contain the greater percentage of badly faulted joints.

The faulting found at transverse cracks was definitely less in magnitude than that at transverse joints, in fact, faulting at cracks was not sufficiently serious to constitute a problem. It is thought that both the longitudinal joint construction and the use of edge-bar reinforcement have been beneficial in maintaining slab alinement at transverse cracks. On the other hand, faulting at joints had in many cases developed to an objectionable extent and there is no doubt that, had there been some suitable structural connection in the transverse joints, the condition that existed would not have developed.

Second conclusion.—From the evidence, it must be concluded that some adequate structural connection between the ends of pavement slabs meeting at a transverse expansion joint is an essential part of a good joint design, even though the slab ends themselves are structurally adequate.

COLLATERAL OBSERVATIONS

Pavement conditions found in the course of the survey led to a number of significant observations not directly related to the primary purpose of the investigation. These are presented as information of interest and value to those concerned with the structural performance of concrete pavements.

Concrete pavements, constructed with expansion joints at intervals of approximately 100 feet and without contraction joints tend to be reduced by transverse cracking, to slab lengths of between 15 and 20 feet and the greater part of this cracking occurs during the first 4 years of pavement life.

If traffic volumes at the time of examination can be accepted as an index of the relative amounts of traffic to which they were subjected, it must be concluded that the amount of transverse cracking that ultimately develops bears little or no relation to the total number of vehicles or to the number of heavy wheel loads that pass over the pavement.

Pavements that contained distributed reinforcement appeared to be in better condition than plain concrete pavements of the same age.

The use of too much steel in the form of tie bars across a longitudinal joint may contribute to longitudinal cracking by restraining warping and by preventing proper adjustment or seating of the pavement on the subgrade in case unequal expansion or settlement occurs in the subgrade.

The defect that has been called an "infiltration crack," although not prevalent, is sufficiently frequent to merit attention. While probably not the result of any single cause, the impacted condition of the expansion joint, particularly near the edge of the pavement, is believed to be an important contributing factor. It appears that these cracks could not be caused solely by load stress.

It is important that expansion joints be maintained in a condition that permits free movement of the slab ends.

Gradual closing of expansion joints with time seems to be primarily the result of gradual opening of the transverse cracks between joints. If the normal functioning of the expansion joints is to be maintained, it appears necessary either to eliminate cracking or to use steel reinforcement to prevent openings at such cracks as form between joints.

There is evidence to indicate that an adequately reinforced concrete pavement slab requires less expansion space than one of equal size without reinforcement.

Expansion failures of the type commonly known as "blow-ups" were not found during this survey, in spite of the fact that frequently the expansion joints were closed or filled with tightly compressed soil. It is possible that the foreign matter in the joints and cracks was sufficiently yielding to prevent this type of failure.

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LONGITUDINAL CRACKING IN CONCRETE PAVEMENTS

A REPORT ON A STUDY OF CRACKING IN KENTUCKY, INDIANA, AND ILLINOIS

By the Division of Physical Research, Public Roads Administration

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SERIOUS LONGITUDINAL CRACKING has occurred in a number of the concrete pavements constructed during the war emergency, some of the most serious of which has been found in Kentucky, Indiana, and Illinois. The pavements on which this cracking occurred are on strategic highways that carried a large amount of heavy war traffic. It was suspected that this cracking might have been caused by the exceptionally heavy traffic or that it might be the result of certain changes in the design of the 1942–43 pavements which it had been necessary to make because of war restrictions.

The cracking was first noted approximately 3 months after the pavements were opened to traffic and each of the three States made independent investigations of the probable cause in the spring or early summer of 1943. It was found that the longitudinal cracking had occurred in panels where there had been a delay in fracturing or actual functioning of the weakened-plane longitudinal joint.

A cooperative investigation to study this condition was started during the summer of 1943 by the Public Roads Administration and the State Highway Departments of Kentucky, Indiana, and Illinois. As a result, two surveys have been made of the pavements included in the investigation, the first during the summer of 1943 when they were approximately 9 months old and the second during the summer of 1944.[1]

This report includes the material collected in the independent investigations by the States and in the cooperative investigation.

It was known from the investigations made independently by the States that serious longitudinal cracking had occurred only in the pavements constructed in 1942. However, it was decided to make a general inspection of a limited number of older pavements of standard design that had been subjected to the same traffic as the 1942 pavements. An inspection was made also of a limited number of 1943 pavements to determine whether there was a delay in the fracturing of the longitudinal joint. The 1943 pavements were parts of 1942 projects which were not completed until the spring of 1943.

The pavements studied in the cooperative investigation together with certain design data and other information are presented in tables 1, 2, and 3.

The Kentucky pavements are located on route U S 31–W between Upton and Kosmosdale. Indiana pavement SN–FA 13–B (2) is located on route U S 40 near Greenfield while Indiana pavements SN–FA 74–A (2) A (3) B (5) and 74–D (5) H (2) are located on route U S 52 between Lebanon and LaFayette. Illinois pavements DA–WI 4–A (1), DA–WI 4–C 4–D, and SN–A–FA 25 (7) (2) are located on U S 66 Alt. a short distance south of Joliet while pavement FA 133–D is located on State route 125 a short distance west of Springfield.

[1] Those participating in the field survey included G. L. Logan, E. L. Lyons, T. R. Thomas, and Everette Gordon of the Kentucky, Department of Highways; R. F. Berns of the Indiana State Highway Commission; F. W. Fullenwider, R. W. Gerling, O. Larsen, E. R. Clemmons, and John Burke of the Illinois Division of Highways; and Mack Galbreath, W. R. Woolley, E. C. Sutherland, D. C. Brooks, P. M. Cassidy, and G. R. Harr of the Public Roads Administration.

WAR CONDITIONS NECESSITATED CHANGES IN DESIGN

A reasonably large range in thickness of pavements is represented in the entire group of projects. None of the 1942–43 pavements was reinforced and the dummy center joint was generally used. The width of the ribbon in the dummy center joint was .25, .33, and .30 of the center depth of the pavement in Kentucky, Indiana, and Illinois, respectively. The depth of the ribbon below the surface varied from slightly below the surface to as much as one-half inch or more.

The specifications for the aggregates, the concrete mixture, and the placing of the concrete for the 1942–43 pavements were the same as those which had been used by the different States during the previous several years. Also the strengths of the concrete, as determined from control specimens, were normal.

The important changes in the design of the 1942–43 pavements as compared to the standard design used previously were:

Kentucky

1. A dummy longitudinal joint, without tie bars, was substituted for one containing a deformed metal plate with tie bars.
2. Distributed reinforcement was eliminated and the cross section was changed from a 9–7–9-inch thickened-edge section to one of 8-inch uniform or a 9–8–9-inch thickened-edge section.
3. Load-transfer devices were eliminated at all transverse joints.

Indiana

1. Distributed reinforcement was eliminated.
2. The spacing of the contraction joints was reduced from 40 to 20 feet.
3. Load-transfer devices at the contraction joints were eliminated.

Indiana has used the dummy longitudinal joint for a number of years.

Illinois

1. A dummy longitudinal joint was substituted for a joint containing a deformed metal plate.
2. Distributed reinforcement in the pavement and load-transfer devices in the transverse joints were eliminated.
3. The thickness of the cross section of the 1942 pavements was greater than that used previously.

No part of the 1942–43 pavements was opened to traffic prematurely. The Indiana pavements were

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TABLE 1.—*Design data on various projects included in the investigation in Kentucky*

Federal-aid project No.	US Route No.	Year built	Length (approximate)	Width	Cross section	Reinforcement	Type of longitudinal joint	Transverse joints: Expansion: Spacing	Expansion: Width	Expansion: Load transfer	Contraction: Spacing [1]	Contraction: Load transfer	Type of coarse aggregate	Method of curing	Average strength of concrete: Compressive 28 days	Average strength of concrete: Transverse 7–14 days
			Miles	*Feet*	*Inches*			*Feet*	*Inches*		*Feet*				*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
SN–FA 169–E (1)	31 W	1942–43	4.4	22	9–8–9	None	Dummy ⅛- x 2-inch ribbon.	120	1	None	20	None	Limestone	Membrane (Aquastatic).	3,720	
			.6	40	8 Unif.											
SN–FA 169–C (2)	31 W	1942–43	.6	22	9–8–9	None	Butt construction joint.	120	1	None	20	None	do	Burlap and Sisal Kraft paper 72 hours.	3,250	
SN–FA 169–F (2)	31 W	1942	7.0	22	9–8–9	None	Dummy ⅛- x 2-inch ribbon.	120	1	None	20	None	do	do	3,530	620
			.2	40	8 Unif.											
DA–WR 3	31 W	1942–43	10.2	22	8 Unif.	None	do	120	1	None	20	None	do	do	3,730	650
			.5	40	8 Unif.											
DA–WR 1	31 W, 60	1942–43	3.9	22 and 44	9–8–9	None	Dummy ⅛- x 2-inch ribbon, tie bars 4 ft. c-c.	500	1	None	20	None	Gravel (Ohio River).	Membrane (Truecure).	3,765	690
FA–79–B (2)	31 W	1937	2.7	20	9–6½–9	43-lb. wire mesh.	Metal center strip, tie bars 4 ft. c-c.	90	¾	¾-inch dowels 12 in. c-c.	30	¾-inch dowels 12 in. c-c.	Limestone	Burlap and straw 7 days.	3,910	
FA–79–D (2)	31 W, 60	1940	8.6	22 and 33	9–7–9	41-lb. wire mesh.	Metal center strip, tie bars 4 ft. c-c.	120	1	Yes	60	Yes	do	Burlap and Sisal Kraft paper 72 hours.	4,100	

[1] Spacing of joints including expansion joints.

TABLE 2.—*Design data on various projects included in the investigation in Indiana*

Federal-aid project No.	US Route No.	Year built	Length (approximate)	Width	Cross section	Reinforcement	Type of longitudinal joint	Transverse joints: Expansion: Spacing	Expansion: Width	Expansion: Load transfer	Contraction: Spacing [1]	Contraction: Load transfer	Type of coarse aggregate	Method of curing [2]	Average compressive strength 6 months
			Miles	*Feet*	*Inches*			*Feet*	*Inches*		*Feet*				*Lbs. in.*$^{-2}$
SN–FA 13–B(2)	40	1942	6.5	22	9–7–9	None	Dummy ⅛ x 2½-inch ribbon, tie bars 5 ft. c–c	120	¾	Dowels 12 inch c–c	20	None	Gravel	Burlap and straw.	5,210
SN–FA 74–A(2), A(3), B(5)	52	1942	8.4	22	9–7–9	None	do	120	¾	Dowels 12 inch c-c. and Translode.	20	None	do	do	6,150
SN–FA 74–D(5), H(2)	52	1942	8.3	22	9–7–9	None	do	120	¾	do	20	None	Mixture stone and gravel.	do	5,400

[1] Spacing of joints including expansion joints.
[2] Wet burlap 1 day and straw 6 days.

TABLE 3.—*Design data on various projects included in the investigation in Illinois*

Federal-aid project No.	US Route No.	Year built	Length (approximate)	Width	Cross section	Reinforcement	Type of longitudinal joint	Traverse joints: Expansion: Spacing	Expansion: Width	Expansion: Load transfer	Contraction: Spacing [1]	Contraction: Load transfer	Type of coarse aggregate	Method of curing [2]	Average transverse bending strength at 14 days
			Miles	*Feet*	*Inches*			*Feet*	*Inches*		*Feet*				*Lbs.*in.$^{-2}$
DA–WI 4–A(1)	66 (Alt.)	1942	4.6	22	10 Unif.	None	Dummy 1/8- x 3-inch ribbon, tie bars 30 in. c–c.	120	¾	None	20	None	Gravel	Paper and straw.	800
DA–WI 4–C 4–D	66 (Alt.)	1942	4.3	22	10 Unif.	None	do	120	¾	None	20	None	Crushed gravel	do	890
SN–A–FA 25 (7) (2)	66 (Alt.)	1942	5.6	22	10 Unif.	None	do	120	¾	None	20	None	Gravel	do	670
FA 133–D	125	1942	3.6	22	10 Unif.	None	do	120	¾	None	20	None	Crusbed gravel	Paper	780

[1] Spacing of joints including expansion joints.
[2] Where paper curing was used tbe period was 72 hours. Straw curing, indefinite period, was used on late season construction.

TABLE 4.—*Summary of subgrade data for Indiana projects*

Project and station	Coarse sand	Fine sand	Silt	Clay	Colloids	Liquid limit	Plasticity index
Project SN-FA 13-B(2):	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
584+00	10	19	44	13	4	37	9
620+00	12	23	38	14	6	26	8
646+00	8	20	52	15	4	33	7
702+00	6	31	38	17	5	30	7
765+00	7	28	44	16	4	26	6
822+00	7	27	42	16	6	24	4
843+00	11	19	41	19	8	37	11
855+00	9	24	29	19	10	21	6
Project SN-FA 74-A(2) A(3) B(5):							
1400+00	4	20	56	13	6	30	9
1456+00	3	17	48	23	8	28	7
1510+00	7	18	50	13	6	28	7
1532+00	3	35	37	20	5	30	12
1560+00	3	14	59	15	9	27	6
1615+00	5	25	47	15	6	32	10
1655+00	6	21	46	21	5	28	5
1703+00	5	25	38	21	8	28	4
1715+00	4	13	51	20	11	39	12
1744+00	3	18	56	17	6	27	3
1774+00	4	19	48	19	9	32	9
1806+00	4	16	55	15	11	37	6
1847+00	7	16	54	11	9	33	8
Project SN-FA 74-D(5) H(2):							
330+00	2	22	35	28	12	28	5
380+00	10	21	41	15	11	40	10
400+00	2	22	50	19	7	30	7
454+00	7	27	42	16	7	32	10
470+00	5	17	46	12	19	33	10
530+00	5	18	54	15	7	33	8
610+00	3	20	54	18	5	31	8
675+00	4	24	48	18	5	32	10

closed to traffic for 21 days after the concrete was placed or until the control specimens showed a transverse bending strength of 550 pounds per square inch. The Illinois pavements were closed to traffic until the control specimens showed a transverse bending strength of 650 pounds per square inch.

SUBGRADE AND DRAINAGE CONDITIONS WERE GENERALLY GOOD

It was not possible for the Kentucky authorities to make a special subgrade study in connection with this investigation. However, it is known from the investigations made prior to construction that there is nothing unusual about the subgrade under the pavement where the longitudinal cracking occurred. The subgrade for these pavements was constructed under the same specifications and with the same care as has been customary in the past. The surface drainage was excellent on all of the pavements investigated in this State.

Certain physical properties of the soils in the subgrades of the various projects investigated in Indiana are shown in table 4. The samples from which these data were obtained were taken at depths of from 0 to 18 inches below the natural ground elevation prior to grading. The soil was of the same type on all three pavements and it was very uniform over the full length of each pavement. Samples taken at depths greater than 18 inches showed the same characteristics as those taken at a less depth.

The type of subgrade under the pavements studied is very common in Indiana and the soils experts of the State could find nothing unusual about the subgrade where longitudinal cracking occurred. The subgrade was constructed under the same specifications and with the same care as had been customary in the past. The surface drainage was excellent on all the pavements investigated. Subgrade drains had been placed at all wet areas found during construction.

On Indiana pavement SN-FA 74-D (5) H (2) a 6- to 8-inch sand-clay base was placed on the northbound lane between the following stations: 409+48—465+50, 493+00—650+00, 680+00—698+00.

TABLE 5.—*Summary of subgrade data for Illinois projects*

Project and station	Sand	Silt	Clay	Colloids	Liquid limit	Plasticity index
Project DA-WI 4-A (1):	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
14+00	56	27	17	5	35	12
69+00	13	58	29	10	35	11
113+00	11	61	25	6	56	25
160+00	12	62	26	7	38	12
209+00	18	65	17	3	44	15
243+00	20	60	20	3	45	19
Project DA-WI 4C, 4D):						
26+00	12	61	26	4	49	16
50+00	14	59	26	7	38	12
83+00	13	58	29	10	35	11
134+00	16	55	29	11	39	14
178+00	13	48	39	12	55	30
210+00	15	46	39	18	41	17
230+00	14	40	46		36	17
295+00	16	59	25	5	46	14
308+00	16	61	22	8	31	9
342+00	9	59	32	8	47	16
359+00	46	36	18	4	30	10
Project SN-A-FA 25 (7) (2):						
250+00	7	71	22	6	41	16
337+00	36	45	18	4	35	13
352+00	20	38	42	16	57	34
Project FA 133-D:						
1+00	6	70	24	10	33	12
16+00	5	60	35	14	42	18
29+00	4	60	36	16	43	22
51+00	6	59	35	17	46	26
127+04	13	70	17	6	27	5

The physical properties of the subgrades on the pavements investigated in Illinois are shown in table 5. These data were obtained from samples taken at depths of from 0 to 18 inches below the natural ground elevation prior to grading. The subgrades for the 1942-43 pavements were constructed under the same specifications used during the previous several years. The surface drainage on all pavements was excellent.

On Illinois pavement DA-WI-4-A (1) a 6-inch crushed-stone base was placed under the pavement. A typical gradation of the material used in the base is as follows:

	Percent
Passing 1-inch sieve	100
Passing ½-inch sieve	78
Passing No. 4 sieve	43
Passing No. 8 sieve	30
Passing No. 16 sieve	20
Passing No. 200 sieve	10

No base was placed under the pavement on Illinois pavement DA-WI-4 C, 4-D. Part of the Illinois pavement project SN-A-FA 25 (7) (2) was laid directly over an old concrete pavement. The old pavement was 18 feet wide while the new pavement is 22 feet wide. The 2-foot overhang on each side was placed on a 6-inch gravel base, a typical gradation of which follows:

	Percent
Passing 3-inch sieve	100
Passing No. 4 sieve	48
Passing No. 50 sieve	12
Passing No. 200 sieve	6

PAVEMENTS WERE SUBJECTED TO HEAVY TRAFFIC

Traffic data on the pavements studied in the three States are shown in table 6.

Kentucky.—The pavements in this State on which the longitudinal cracking developed are located in the vicinity of Fort Knox and carry a large amount of heavy truck traffic. While not indicated in the table, Kentucky pavements DA-WR 3, FA 79-B (2), FA

TABLE 6.—*Traffic data on pavements studied*

Kentucky				Indiana		Illinois				
Project	Average daily traffic in 1942	Project	Average daily traffic in 1942	Project	Average daily traffic in 1943	Project	Average daily traffic in 1943	Commercial traffic—		
								Time of survey	Average daily	Heavy daily
SN–FA 169–E (1)	875	FA 79–B (2)	6,300	SN–FA 13–B (2)	1,690	DA–WI 4–A (1)	3,520	1942	650	65
SN–FA 169–C (2)	1,475	FA 79–D (2)	6,500	SN–FA 74–A (2) A (3), B (5).	1,550	DA–WI 4–C, 4–D	3,520	1942	650	65
SN–FA 169–F (2)	2,000	DA–WR 1	8,000	SN–FA 74–D (5) H (2)	1,550	SN–A–FA 25–(7) (2)		1941	870	550
DA–WR 3	7,250					FA 133–D		1941	400	40

TABLE 7.—*Summary of crack survey data on the Kentucky projects*

Project and station	Panels in which longitudinal cracking occurred			
	1943		1944	
	Number	*Percent*	*Number*	*Percent*
Project SN–FA 169–F (2):				
829—840	1	2	2	4
840—850	1	2	4	8
850—860	2	4	3	6
860—870	0	0	0	0
870—880	0	0	0	0
880—890	3	6	4	8
890—900	0	0	3	6
900—910	0	0	0	0
910—920	0	0	6	12
920—930	1	2	1	2
930—940	2	4	5	10
940—950	2	4	5	10
950—955	14	56	14	56
955—960	1	4	1	4
960—965	3	12	3	12
965—970	6	24	6	24
970—975	0	0	0	0
975—980	5	20	5	20
980—985	1	4	1	4
985—990	6	24	6	24
990—995	4	16	8	32
995—1,000	5	20	5	20
1,000—1,001+70	1	12	1	12
1,001+70—1,008+25	3	5	3	5
Project DA–WR 3:				
0—28 [1] (two left lanes)	0	0	0	0
0—28 (two right lanes)	42	30	44	31
28—81	222	84	222	84
81—134	150	57	150	57
134—186	46	17	50	19
186—239	70	26	80	30
239—292	66	25	70	26
292—345	63	24	63	24
345 (2 miles) to 450	145	27	145	27
450—503	2	1	6	2
503—556	6	2	6	2
557—567 [2]	0	0	0	0

[1] Four-lane pavement between stations 0 and 28.
[2] A separate 11-foot lane between stations 557 and 567.

79–D(2), DA–WR 1, and part of SN–FA 169–F have been used for testing tanks from Fort Knox. These tanks weigh approximately 32 tons and travel at 20 to 25 miles per hour. Until about 6 months before the survey the tank treads were equipped with rubber pads but after that time many of them were equipped with steel lugs.

The war emergency load limit in Kentucky was 28,000 pounds gross load with a maximum axle load of 16,000 pounds.

Indiana.—In a survey on Indiana pavement SN–FA 13–B (2), taken over five 8-hour days, it was found that 24.5 percent of the traffic was commercial and an average of 29 trucks, per 8-hour day, exceeded 30,000 pounds gross weight. Two overloaded trucks, one weighing 60,000 pounds and the other 57,000 pounds, were noted during the 5-day period.

In an 8-hour survey, August 1943, on State route 52 between Indiana pavements SN–FA 74–A (2) A (3) B (5) and SN–FA 74–D (5) H (2) it was found that 29 percent of the traffic was commercial. The gross weights of the 10 heaviest trucks were: 55,000, 51,600, 47,200, 46,200, 45,400, 44,800, 44,400, 42,000, 40,200, and 40,000 pounds.

The maximum legal axle load in Indiana is 18,000 pounds. The number of overloaded trucks on the pavements during the war greatly exceeded those found in normal times.

Illinois.—The number of commercial and heavy vehicles on the various Illinois pavements are shown in table 6. The maximum axle load in this State is 16,000 pounds. The maximum gross loads for various types of vehicles were:

	Pounds
Vehicle with 2 axles	24,000
Vehicle with 3 axles	40,000
Truck and trailer combination with at least 4 axles	56,000

No army tanks used the pavements on which longitudinal cracking occurred in either Indiana or Illinois.

The ribbon placed in the pavement to form the dummy longitudinal joint was generally placed a short distance below the surface. It was found from cores drilled at the center joint that the presence or absence of a crack over the ribbon was an indication of whether or not the longitudinal joint had fractured. For this reason the presence or absence of such a crack was noted in the crack surveys. The core data will be presented later in this report.

CRACK SURVEY DATA IN KENTUCKY

Kentucky pavement SN–FA 169–E (1).—The surface studied was laid in the spring and early summer of 1943. At the time of the first survey this pavement had been subjected for a short time to heavy truck traffic but no army tanks had passed over it regularly. Longitudinal cracks were not found in the first survey, and it was estimated that the longitudinal joint had not fractured in approximately 40 percent of the panels. In the second survey three full-length and three part-length cracks were found and the center joint still had not fractured in 10 percent of the panels.

Kentucky pavement SN–FA 169–C (2).—This pavement was constructed in two separate 11-foot lanes with a butt longitudinal joint between them. It had been subjected to heavy truck traffic, but had not been used by army tanks. Longitudinal cracking had not occurred up to the time of the last survey.

Kentucky pavement SN–FA 169–F (2).—This pavement carried a large amount of heavy truck traffic throughout its full length. Between stations 950 and 1008 it had been used by a large number of medium army tanks, but few if any army tanks had used the pavement between stations 829 and 950.

The data obtained in the two surveys are summarized in table 7. The word "panel" as used in this and succeeding tables applies to the pavement width of two

lanes and the length between successive transverse joints. In the analysis of the crack survey data a crack extending three-quarters or more of the length of the panel was tabulated as a full-length crack while one of lesser length was tabulated as a part-length crack.

At the time of the first survey in 1943, a total of 2 percent of the panels contained full- or part-length longitudinal cracks between stations 829 and 950 and 15 percent between stations 950 and 1008+25. In the survey of 1944, a total of 5 percent of the panels were cracked between stations 829 and 950, and 16 percent between stations 950 and 1008+25. Fifty-two percent of the cracks in this surface were full length. In the first survey only one or two longitudinal cracks were present in panels where the center joint had fractured, but in the second survey there was a total of eight such cracks. Ten panels were found in the second survey in which the center joint had not fractured and in which no longitudinal cracks had occurred.

It is very possible that the greater amount of longitudinal cracking between stations 950 and 1008+25 as compared to that between stations 829 and 950 was due to the stresses caused by the army tanks.

Kentucky pavement DA–WR 3.—This was a four-lane pavement between stations 0 and 28. The four lanes were divided at the center with a butt longitudinal joint. The longitudinal joint at the center of the two right lanes, in the direction of stationing, was of the dummy type while that between the two left lanes was of the deformed metal plate type. The pavement studied between stations 557 and 567 was a single lane laid along the edge of an older pavement. The remainder of the pavement, on this project, was two lanes with a dummy longitudinal joint. This pavement had been subjected to heavy truck and tank traffic throughout its full length.

Table 7 shows that there was no longitudinal cracking in those parts of the pavement with a definite separation at the longitudinal joint at the time of construction; that is, in the two left lanes between stations 0 and 28 and in the single-lane construction between stations 557 and 567. Another notable condition shown by the data of table 7 is the variation in the amount of longitudinal cracking between the two ends of the pavement. There appeared to be no differences between the subgrades at the two ends that would explain this variation and the whole pavement has been subjected to the same traffic. It is probable that the variation in amount of cracking was caused by differences in weather when the different parts of the pavement were laid. Construction was started in the summer at station 556 and the pavement was not completed until very late in the fall.

There was little difference in the amount of cracking observed in the two surveys. In the 1944 survey, 63 percent of the cracks were full length, 10 panels were found with longitudinal cracks on both sides of the longitudinal joint, 16 in which longitudinal cracks had occurred where the center joint had fractured, and 15 in which no longitudinal cracks had occurred although the center joint had not fractured.

Kentucky pavement DA–WR 1.—Only one survey, that of 1943, was made on this pavement. At that time a study was made of 1.8 miles of four-lane pavement and a total of 3.2 miles of single-lane construction. The four-lane pavement was divided at the center with a butt construction joint, and a dummy longitudinal joint was placed between the two lanes on each side. The single-lane portion of the pavement was placed as widening along the two edges of an older pavement. The four-lane portion of the pavement and one-half of the single-lane construction (left lane) had been subjected to heavy truck and tank traffic for approximately 10 months prior to the survey. The other half of the single-lane pavement (right lane) was not completed until the spring of 1943 and had been subjected to the heavy traffic for a period of only approximately 2 months prior to the survey.

FIGURE 1.—LONGITUDINAL CRACK IN SECTION OF PAVEMENT WHERE THE CENTER JOINT HAS NOT CRACKED.

Of the 1,148 panels examined in the four-lane pavement, 220, or 19 percent, contained full- or part-length longitudinal cracks. There were no longitudinal cracks in the single-lane pavement.

Kentucky pavement FA 79–B (2) and 79–D (2).—These two pavements were of standard design and were laid between 1937 and 1941. They were included in the study because, since 1942, they had been under the same traffic as the 1942 pavements in which serious longitudinal cracking occurred. Both pavements were reinforced and the longitudinal joint was of the metal plate type. No longitudinal cracking was found in these two pavements.

DELAY IN FRACTURING CENTER JOINT AN IMPORTANT CAUSE OF CRACKING

In the 1943 survey all but one or two of the longitudinal cracks were found in panels where the dummy center joint had not fractured and therefore was not functioning as a longitudinal joint. Figure 1 shows a typical example of this condition.

Cores were drilled at the center joint in a number of panels where there was some question as to whether the

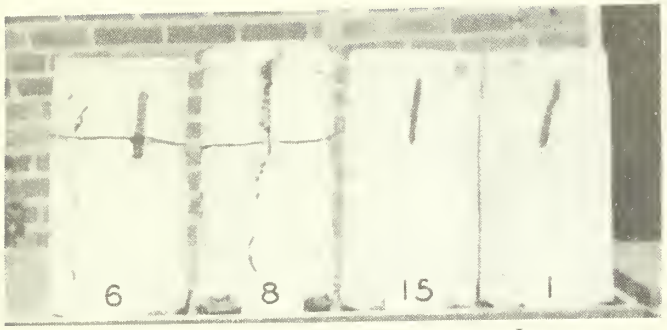

FIGURE 2.—TYPICAL CORES DRILLED AT THE LONGITUDINAL JOINT IN THE 1942 KENTUCKY PAYMENTS.

joint had fractured. Several of these typical cores are shown in figure 2. These cores show that the center joint had not fractured below the ribbon except in the one case in which a crack had appeared above the ribbon. The panel from which core 6 was taken contained a full-length longitudinal crack at a distance varying from 2 to 18 inches from the center joint, the distance being approximately 2 inches at the point where the core was drilled. The longitudinal joint had fractured in the panel from which core 8 was drilled and there was no longitudinal crack in this panel. Cores 1 and 15 were taken from panels in which full-length longitudinal cracks had occurred and, as indicated, the longitudinal joint had not fractured in these panels.

These cores show that the top edge of the ribbon was, in some cases, an inch or more below the surface of the pavement. It was thought that this might have contributed to the delay in fracturing of the joint. However, it was found in the field study that the center joint had not fractured and that longitudinal cracks had developed in many panels in which the top edge of the ribbon was near the surface.

The amount of longitudinal cracking appeared to be approximately the same in cuts and fills and followed no definite pattern either in the same or between different projects. For example, in a typical section of pavement the distance of the cracks from the center joint was as follows:

	Percent
0–12 inches	31
12–24 inches	60
24–36 inches	8
More than 36 inches	1

The part-length cracks varied from 2 to 15 feet in length and they generally broke from the longitudinal joint at some interior point and continued the remaining length of the panel. Some full-length cracks that remained fairly close to the longitudinal joint actually passed from one side of the joint to the other at two or three points within the length of the panel.

Many of the cracks had opened up badly because of the absence of reinforcement in the pavement and of tie bars in the longitudinal joint. Figure 3 shows several typical examples of cracking in the Kentucky pavements laid in 1942.

Mr. E. L. Lyons of the Kentucky Department of Highways observed that a greater amount of longitudinal cracking developed in the parts of the pavement laid late in the year than in the parts laid earlier. This condition was mentioned in the discussion of Kentucky pavement DA–WR 3, where a serious amount of longitudinal cracking developed in the parts of the pavement laid during October and November while

TABLE 8.—*Summary of the crack survey data on the Indiana projects*

Projects and stations	Time pavement was laid	Panels in which longitudinal cracking had occurred [1]			
		1943		1944	
		Number	*Percent*	*Number*	*Percent*
Project SN–FA 13–B (2):					
582—599	November	19	22	20	24
599—625	Late October	56	43	56	43
625—651	Middle October	44	34	44	34
651—677	do	68	52	68	52
677—704	Early October	12	9	12	9
704—729	Late September	1	1	3	2
729—758	do	0	0	1	1
758—782	Middle September	5	4	5	4
782—808	do	2	2	2	2
808—835	do	0	0	0	0
835—861	Early September	3	2	3	2
861—887	do	0	0	0	0
887—909	Late August	2	2	2	2
Project SN–FA 74–A (2) A (3) B (5):					
Two north-bound lanes:					
1661—1734	Early October	21	6	28	8
1734—1782	Middle October	2	1	6	2
Two south-bound lanes:					
1395—1424	Middle September	0	0	8	6
1424—1450	Early September	34	26	44	34
1450—1490	do	5	2	6	3
1490—1540	Late August	8	3	9	4
1540—1605	Middle August	14	4	19	6
1605—1650	Early August	1	0	3	1
1650—1751	July-August	1	0	0	0
1751—1834	July	0	0	1	0
1834—1861	Late September	4	3	5	4
Project SN–FA 74–D (5) H (2):					
Two south-bound lanes:					
312—410	Early August	0	0	5	1
Two north-bound lanes:					
320—407	Late July	11	3	11	3
407—538	Early July	3	0	12	2
538—618	Late June	5	1	5	1
618—671	do	4	2	5	2
671—731	Middle June	0	0	2	1

[1] Two cracks were counted in panels where cracks had occurred on each side of the longitudinal joint.

only a small amount occurred in the parts laid earlier. (See table 7.) This was found to be true of other 1942 pavements. It appears that there is a greater delay in the fracturing of the longitudinal joint in those parts of the pavement laid late in the year and this delay increases the chances for subsequent longitudinal cracking. The possible effect of the time of the year at which the pavement was laid on the development of this form of cracking was investigated more thoroughly in Indiana and Illinois.

While this investigation was primarily a study of longitudinal cracking it was observed that in Kentucky the army tanks with the steel lug treads caused serious surface damage to the pavement and spalling at the transverse joints. There had been no transverse cracking to speak of in the 1942 pavements investigated and only a normal amount in the older pavements of standard design.

Conclusions that may be made as a result of the investigation of Kentucky pavements are:

1. There is a serious delay in fracturing of the dummy center joint as it has been constructed. This appears to be especially true of pavements laid late in the year.

2. The heavy traffic has not caused longitudinal cracking in those parts of the pavement in which there was a definite separation of the longitudinal joint at the time of construction.

3. There is a greater percentage of longitudinal cracking in the parts of the 1942 pavements used by army tanks than in the other parts.

FIGURE 3.—TYPICAL LONGITUDINAL CRACKS IN THE 1942 KENTUCKY PAVEMENTS.

CRACK SURVEY DATA IN INDIANA

An intensive study was made of three pavements in Indiana laid in 1942. Serious longitudinal cracking had developed in only one of these pavements and it was first detected approximately 3 months after the pavement was opened to traffic.

The dummy center joint has been used in many pavements in this State for a number of years. Since the longitudinal cracking appeared to be associated with the dummy center joint an inspection was made of several older pavements, built with this type of center joint, in order to study the condition of the joint and determine whether or not there had been any longitudinal cracking in pavements laid prior to 1942. The crack-survey data for the three 1942 pavements studied are summarized in table 8.

Indiana pavement SN–FA 13–B (2).—Little difference was found in the amount of longitudinal cracking on this surface as shown by the 1943 and 1944 surveys, the percentage of cracked panels being approximately 13 percent in each case. The amount of longitudinal cracking varied greatly between the two ends of this pavement, however. In the part laid during October and November an average of 33 percent of the panels contained full- or part-length cracks while in the part laid during August and September only a small amount of cracking appeared. Seventy-one percent of the cracks in this pavement are full length.

In the 1943 survey only one or two longitudinal cracks were found in panels having a fractured center joint. However, in the 1944 survey 56 such cracks were found. On this pavement many of the longitudinal cracks were 2 feet or more from the center joint and this may explain why the center joint fractured in so many panels in which longitudinal cracks had already occurred. At the time of the 1944 survey all of the panels in which the center joint had not fractured contained longitudinal cracks. From this it may be expected that little additional longitudinal cracking will develop in this pavement.

Several cores drilled at the center joint of panels in this pavement are shown in figure 4. The core at the left was taken from a panel with a full-length longitudinal crack approximately 3 feet from the center joint. There was no crack above the ribbon and the joint had not fractured. The center core showed evidence of cracking and there was no longitudinal crack in the panel. The panel from which the right core was taken contained a full-length crack approximately

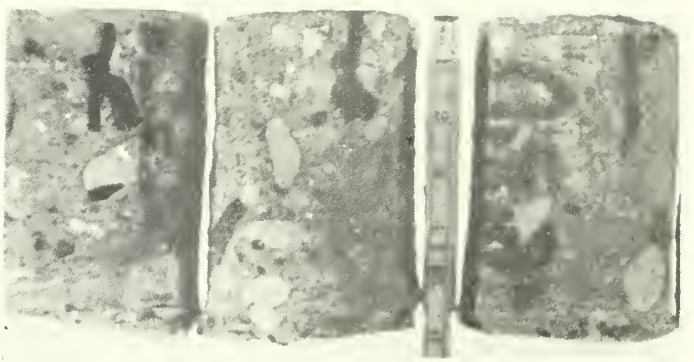

FIGURE 4.—TYPICAL CORES DRILLED AT THE LONGITUDINAL JOINT IN THE 1942 INDIANA PAVEMENTS.

3 feet from the center joint. There was no crack above the ribbon and the joint had not fractured.

The cracks in this pavement were found at distances varying from a few inches to as much as 4 feet from the center joint. Those near the center joint were held closed by the tie bars. The cracks may be divided according to form, into two types, as follows:

1. Those which extended the full length of the panel and were generally approximately parallel to the center joint. As a rule these cracks were 12 inches or more from the center joint.

2. Those which broke from the center joint, at some intermediate point and continued through the remaining length of the panel. These cracks tended to remain fairly close to the center joint and they vary in length from approximately 2 to 15 feet. Approximately 30 percent of the total number of cracks were of this type.

Usually longitudinal cracking developed on only one side of the longitudinal joint. In a short section, however, between stations 582 and 704 longitudinal cracking had occurred on both sides of the longitudinal joint in 23 panels. This is the section containing the greatest amount of longitudinal cracking and the cracks are 2 feet or more from the center joint. Figure 5 shows typical longitudinal cracks in this pavement in 1943 while figure 6 shows the condition of some of the cracks observed in 1944.

The traffic was approximately the same over the full length of the section and an examination of the subgrade data, table 4, indicates that there was no important difference in the subgrade between that part of the pavement where there was considerable longitudinal cracking and the part which had a negligible amount. It is possible, however, that there may have been some

FIGURE 5.—TYPICAL LONGITUDINAL CRACKS OBSERVED IN 1943 IN INDIANA PAVEMENT SN–FA 13–B (2).

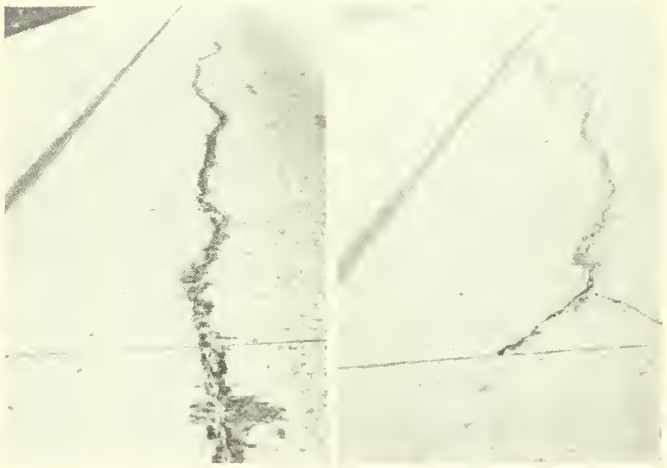

FIGURE 6.—TYPICAL LONGITUDINAL CRACKS OBSERVED IN 1944 IN INDIANA PAVEMENT SN–FA 13–B (2).

variation in the density or moisture condition of the subgrade.

The fact that the longitudinal cracking developed mainly in those parts of the pavement laid late in the construction season may suggest that this part of the pavement was opened to traffic at an earlier age than the other parts. The investigators were informed that this was not true as the different parts of the pavement were opened to traffic as construction progressed.

Part of this pavement was laid during the summer and fall of 1943, construction being completed the first week in November. The cross section of this part of the pavement was 9–8–9 inches, but the depth of the ribbon in the center joint was proportionately the same as that in the 1942 pavement or one third the center depth of the pavement. In the 1944 survey of this part of the pavement no off-center cracking was observed and it appeared that the center joint had opened throughout the full length.

It is difficult to explain the difference in the performance of the part of the pavement laid in 1943 as compared to that laid in 1942. Longitudinal cracking was found in 9–8–9-inch thickened edge and 10-inch uniform thickness pavements in Kentucky and Illinois which indicates that the increase in thickness of the 1943 Indiana pavement, as compared to the 1942 pavement, probably was not responsible for the elimination of the longitudinal cracking. It was pointed out by one of the engineers of the State Highway Commission that the shoulders on the 1942 pavement were in poor condition during the winter of 1942–43 and that the shoulders and subgrade near the edges of the pavement were very wet. The fall of 1943 was comparatively dry and the shoulders were smoothed down so that drainage away from the pavement was obtained soon after construction was completed. It was also pointed out that the 1942 pavement was used by two-way traffic during the winter of 1942–43 while the 1943 pavement was used only by one-way traffic after being completed.

Indiana pavement SN–FA 74–A(2) A(3) B(5).—The longitudinal cracking in this pavement is not serious. In the 1943 survey it was found that 3 percent of the panels contained some form of longitudinal cracking while at the time of the 1944 survey the value had increased to 4 percent. More than 90 percent of the cracks were part length ranging from 2 to 8 feet in length and, as a rule, they were not more than 6 to 12 inches from the center joint.

The percentage of longitudinal cracks was greater in the parts of this pavement laid during September and October than in the parts laid earlier. All of the cracks were in panels or parts of panels in which the center joint had not fractured and at the time of the last survey longitudinal cracking had occurred in all of the panels where the center joint had not fractured. It may be expected, therefore, that little or no additional longitudinal cracking will occur in this pavement. The majority of the cracks were within the limits of the tie bars and were being held closed. Figure 7 shows typical longitudinal cracks in this pavement. From visual inspection the cracks appeared to

FIGURE 7.—TYPICAL LONGITUDINAL CRACKS IN INDIANA PAVEMENT SN–FA 74 A (2), A (3), B (5).

be in approximately the same condition at the time of the 1944 survey as they were in 1943.

Indiana pavement SN–FA 74–D (5), II (2).—Only 2 full-length and 38 part-length cracks were found in this pavement. The center joint was open in all the panels that were free from longitudinal cracking so that there was good reason to expect that no additional longitudinal cracking would develop. The form of the cracking in this pavement was very similar to that described on the previous project. This pavement was constructed during warm summer weather, construction being completed early in August and this may explain why there was less longitudinal cracking than on the two surfaces previously discussed.

LONGITUDINAL CRACKING IN OLDER PAVEMENTS STUDIED

Referring to table 4 it appears that there was no difference between the subgrade on those parts of Indiana pavements SN–FA 74–A (2) A (3) B (5) and 74–D (5) H (2) that showed longitudinal cracking and in those which had no cracking. Also the subgrade on these two projects is of the same type as that on Indiana pavement SN–FA 13–B (2) where serious longitudinal cracking developed.

The question naturally arises as to why the dummy center joint caused difficulty in the 1942 pavements after it had been apparently satisfactory in Indiana and elsewhere. Older pavements with this type of joint were examined in an effort to answer this question. Two pavements, Indiana FAP 74 laid in 1939

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FIGURE 8.—LONGITUDINAL CRACKING IN THE OLDER REINFORCED PAVEMENTS WITH DUMMY CENTER JOINT LAID IN INDIANA IN 1937 (74–E).

and FAP 74–E laid in 1937, were selected for this study. The cross section of these two pavements is 9–7–9 inches, both were reinforced with welded fabric and both had been subjected to heavy truck traffic.

It was found that a small number of longitudinal cracks have formed in these pavements. The majority of these break from the center joint at some intermediate point and continue the remaining length of the panel and, as found in the 1942 pavements, the center joint had not fractured in the panels or parts of panels where the longitudinal cracking developed. Figure 8 shows typical longitudinal cracks found in these pavements.

Although the longitudinal cracking is not prevalent in the older pavements built with the dummy center joint, it is present in sufficient amount to indicate a definite tendency toward such cracking and there is evidence to indicate that it was caused by the failure of the center joint to fracture promptly as intended. This cracking undoubtedly would have caused more concern if the pavements had not been reinforced.

It will be recalled that while the medium army tanks had apparently caused a slight increase in the amount of longitudinal cracking in the 1942 Kentucky pavements they had not caused longitudinal cracking in the older pavements of standard design with a definite separation at the longitudinal joint. In Indiana there were three older concrete surfaces of standard design in the northwestern part of the State that had been used more than a year for testing this same class of tanks and it was decided to make a general inspection of these pavements for such additional information as they might provide on this question.

Two of these surfaces, Indiana FA 32–C and FA 17–A, are divided four-lane pavements laid between 1937 and 1939 and have a combined length of approximately 15 miles of two-lane pavement. The cross

TABLE 9.—*Summary of the crack survey data on the Illinois projects*

Projects and stations	Time pavement was laid	Panels in which longitudinal joint has not broken and no longitudinal cracking has occurred				Panels in which longitudinal cracking has occurred			
		1943		1944		1943		1944	
		No.	Pct.	No.	Pct.	No.	Pct.	No.	Pct.
Project DA-WI 4-A (1) (Left lane):									
13+68—44+31	Middle October	28	18	25	16	1	1	2	1
44+31—82+48	Late October	62	33	47	25	7	4	8	4
82+48—103+94	Early November	8	8	8	8	9	8	9	8
Project DA-WI 4-C, 4-D (Right lane):									
16+21—57+44	Late October	102	49	1	0	10	5	10	5
57+44—71+61	Early November	70	100	0	0	0	0	0	0
Project DA-WI 4-C, 4-D (Left lane):									
49+36—101+26	Middle October	142	54	110	42	9	3	11	4
101+26—164+49	Early October	121	38	68	21	10	3	11	3
164+49—216+31	Late September	133	53	80	32	2	1	3	1
216+31—272+36	Middle September	113	41	90	32	2	1	3	1
272+36—308+82	Early September	41	23	28	15	0	0	0	0
308+82—351+12	Late August	20	10	6	3	6	3	7	3
351+12—451+04	Middle August	210	42	127	26	9	2	9	2
Project SN-A 25 (7) (2): [1]									
49+80—80+30	Late October	18	12	8	5	12	8	12	8
80+30—95+57	Early November	24	32	9	12	10	13	10	13
95+57—106+12	Middle November	7	13	1	2	6	11	6	11

[1] This pavement was placed over an old pavement between stations 49+80 and 95+55.

section of each pavement is 9–7–9 inches, both are reinforced with welded fabric, and have the dummy longitudinal joint.

A total of 12 full- or part-length longitudinal cracks were found in the two pavements. These cracks had the appearance of being old, indicating that they did not occur while the tanks were using the pavement. They were all near the longitudinal joint and were being held closed by the tie bars and reinforcement.

It was observed that the heavy traffic, including army tanks, was causing pumping at many of the expansion joints of these pavements. At a number of joints transverse cracks had occurred 6 to 8 feet from the joint and the intervening short slab was depressed; in some cases sufficiently to require patching.

The third pavement, Indiana NRHM 69–H, laid in 1935, consisted of two 11-foot lanes separated by an older pavement. The outer lanes were of 9–7–9-inch cross section reinforced with a light wire fabric. There were no longitudinal cracks in this pavement. The expansion joints were spaced at intervals of 80 feet with one intermediate contraction joint. Open transverse cracks, at which the steel was apparently broken, were present in a large number of these panels and bad faulting was noted at many of these cracks. Appreciable faulting had not occurred at the transverse joints, however, apparently due to the presence of dowels in the joints.

The conclusions drawn from the investigation of the Indiana pavements are:

1. There is a delay in the fracturing of the dummy center joint sometimes for as much as a year or more. This appears to be especially true of pavements laid late in the year.

2. The longitudinal cracks have formed in parts of the pavement where there was a delay in the fracturing or actual functioning of the center joint.

3. The heavy war traffic, including medium army tanks, has not caused longitudinal cracking in the older pavements of standard design with the dummy center joint. The dummy joint was functioning in these pavements at the time of the survey.

CRACK SURVEY DATA IN ILLINOIS

Longitudinal cracking has occurred in the surfaces of four 1942 projects in this State. In the majority of the surfaces, the cracking was first noted approximately 3 months after they were opened to traffic.

The summary of the crack survey data on the pavements investigated in Illinois is presented in table 9. An extra column has been added to the table to show the number of panels in which the center joint had not fractured and no longitudinal cracking had developed. The number of such panels was much greater in the Illinois pavements than in those of Kentucky and Indiana.

Illinois pavement DA–WI 4–A(1).—At the time of the 1943 survey it was found that 22 percent of the panels had neither fractures of the center joint nor longitudinal cracking. In the 1944 survey the percentage had decreased to 18. Longitudinal cracking was found in slightly less than 4 percent of the panels at the time of the 1943 survey and slightly more than 4 percent at the time of the 1944 survey. Approximately 30 percent of the cracks were full length.

Illinois pavement DA–WI 4–C, 4–D.—The number of panels in this surface with neither fracture of the center joint nor longitudinal cracking amounted to 42 percent in the 1943 and 23 percent in the 1944 survey. Longitudinal cracking was found in approximately 2 percent of the panels in 1943 and 2½ percent in 1944. Approximately 14 percent of the cracks were full length.

Illinois pavement SN–A–FA 25(7) (2).—The part of this pavement between stations 49+80 and 95+57 is resurfacing placed on an old concrete pavement while the remainder was laid on the natural subgrade. In the 1943 survey it was found that there was longitudinal cracking in 10 percent of the panels in the resurfacing and in 11 percent of those laid on the natural subgrade. Approximately 35 percent of the cracks were full length and there was no appreciable increase in the number of cracks in 1944 compared to 1943.

The number of panels, over the full length of the section studied, in which the center joint had not fractured and in which there was no longitudinal cracking, amounted to 17 percent in the 1943 survey and 6 percent in the 1944 survey.

Illinois pavement FA 133–D.—One crack survey was made on this surface during the fall of 1943. Only a small number of longitudinal cracks were found. At the age of 1 year of the 605 panels examined, the center joint had not fractured in 95 or 15 percent of the panels.

As in the other States, cores were drilled at the longitudinal joint in a number of panels to disclose the condition of the joint, especially in those panels where surface inspection indicated that the joint had not fractured. Figure 9 shows several typical cores.

THICKNESS OF ILLINOIS PAVEMENTS A FACTOR IN LONGITUDINAL CRACKING

Core A of figure 9 was taken from a panel with a full-length longitudinal crack approximately 2 feet from the center joint. The top edge of the ribbon was one-half inch below the surface and there was no crack either above or below the ribbon. There was no crack in the

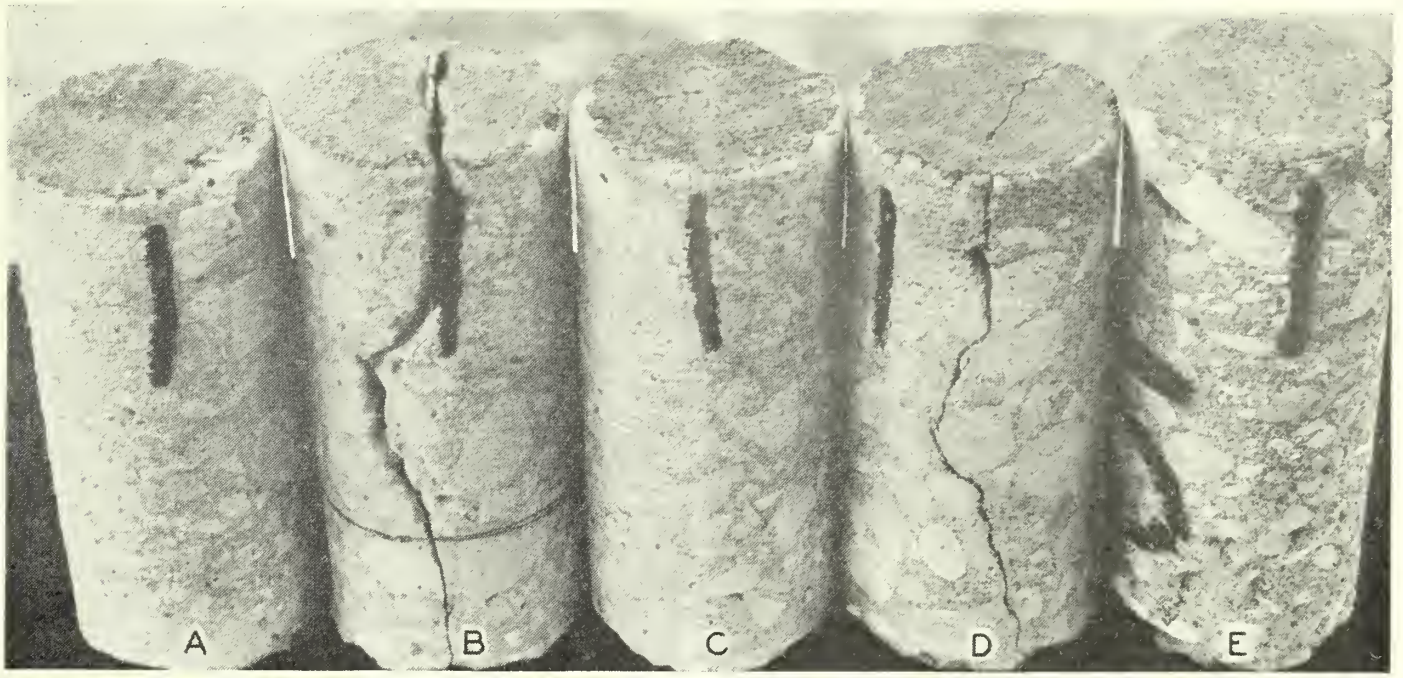

FIGURE 9.—TYPICAL CORES DRILLED AT THE LONGITUDINAL JOINT IN THE 1942 ILLINOIS PAVEMENTS.

panel from which core B was taken. The ribbon was seven-sixteenth inch below the surface and the center joint had fractured. The panel from which core C was taken showed a full-length longitudinal crack. The ribbon was one-fourth inch below the surface and there was no crack either above or below the ribbon. The panel from which core D was taken was cracked. The ribbon was one-half inch below the surface and there was no crack either above or below the ribbon. The panel from which core E was taken contained a full-length longitudinal crack. The ribbon was one-half inch below the surface and there was no crack above or below the ribbon.

As seen from the above data the ribbon forming the center joint was frequently found one-half inch below the surface. Out of 19 cores taken at the center joint the ribbon was within one-fourth inch or less of the surface in 5 cores. The center joint had not fractured, at the time of the 1943 survey, in three of the panels from which these five cores were taken and longitudinal cracking had developed in two of the three panels.

Many of the cracks in the Illinois pavements have formed fairly close to the longitudinal joint and these are being held closed by the tie bars in the joint. In a number of cases, however, the cracks are 2 feet or more from the center joint and it is expected that these cracks will open and perhaps seriously affect the condition of the pavements. Figure 10 shows several typical cracks in the Illinois pavements. The pattern of cracking was approximately the same on the resurfacing as on the other parts of the pavement. With one exception the longitudinal cracks are in panels in which the center joint had not fractured. There were no cases of longitudinal cracking on both sides of the longitudinal joint.

In studying the data from the pavements in Illinois it was noted that there were a large number of panels in which the center joint had not fractured and in which there was no longitudinal cracking. Only a small number of such panels were found in the pavements investigated in Kentucky and Indiana. This difference is attributed to the greater thickness of the Illinois pavements. The cross section of the Illinois pavements in which longitudinal cracking has developed is 10-inch uniform thickness while in Kentucky and Indiana the cross sections are 9–8–9 and 9–7–9 inches respectively.

A study of the data from the Illinois pavements indicates that in those parts of the pavement in which the center joint had not fractured and there has been no longitudinal cracking the tendency has been for the center joint to fracture rather than for longitudinal cracking to occur. This is shown by the fact that during the interval between the two surveys there was only a small increase in the number of longitudinal cracks while, there was a marked decrease in the number of panels with an uncracked center joint and no longitudinal cracking.

As was the case with the pavements investigated in Kentucky and Indiana the amount of longitudinal cracking in the Illinois pavements varies with the time of the year at which the pavement was laid. In the three surfaces included in table 9 the number of panels in which longitudinal cracking was found, at the time of the 1943 survey, amounted to 2, 1, 4, and 8 percent, respectively, in the parts of the pavement laid during August, September, October, and November.

The subgrade data for the Illinois pavements, table 5, does not show any important differences in the characteristics of the subgrade for the parts of the pavement with different amounts of longitudinal cracking. In this connection it will be recalled also that on Illinois project SN–A–FA 25 (7) (2) the amount of longitudinal cracking was approximately the same on the resurfacing as on the other part of the pavement laid on the natural subgrade. The surface drainage appeared to be excellent on all the pavements studied and only a negligible amount of transverse cracking was observed.

In the older Illinois pavements of standard design,

FIGURE 10.—TYPICAL LONGITUDINAL CRACKS IN THE 1942 PAVEMENTS OF ILLINOIS.

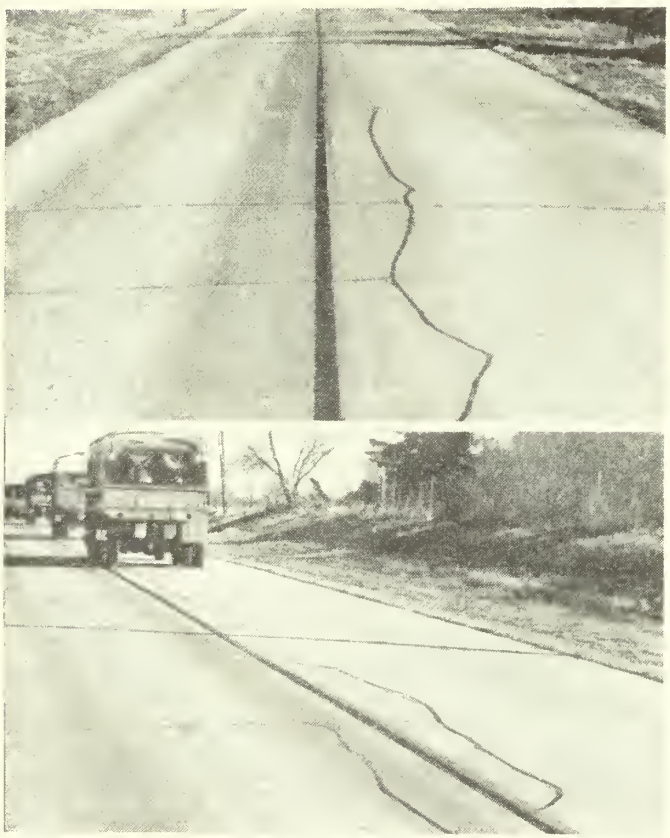

FIGURE 11.—LONGITUDINAL CRACKING IN NEBRASKA PAVEMENTS WITH DUMMY CENTER JOINT, LAID IN 1935–36.

that have been subjected to the same traffic conditions since 1942 as the 1942 pavements studied, longitudinal cracking has not developed. In these, complete partition along the center line was obtained with a deformed metal plate.

The conclusions that may be drawn from the investigation of the Illinois pavements are much the same as those indicated by the examination of the Kentucky and Indiana pavements except as affected by the greater thickness of the Illinois pavements. The delay in the fracturing of the center joint was greater in the case of the Illinois pavements and it was found in pavements laid during warm summer weather as well as in those laid late in the year. There was also a greater delay in the development of longitudinal cracks in the panels in which the center joint had not fractured.

While it was necessary to limit the scope of the investigation to Kentucky, Indiana, and Illinois, the type of longitudinal cracking found in these States has been observed in certain pavements of other States in which the dummy center joint has been used. There are probably few instances where it has been as pronounced as in the 1942 pavements described in this report but there is little doubt that a definite tendency toward this type of cracking exists with the dummy center joint. The greater amount of heavy traffic during the war may have increased this tendency, while the more limited use of reinforcement in the pavements and in some cases the elimination of tie bars in the center joints increased the seriousness and probably permitted a more rapid development of cracking. Figure 11 shows typical longitudinal cracks observed in certain pavements in Nebraska. The cross section of these pavements is 9–7–9 inches and the center joint was formed with a ⅜- by 2½-inch premolded bituminous strip.

DEFINITE SEPARATION AT CENTER JOINT AT TIME OF CONSTRUCTION FOUND NECESSARY

This survey leads to the conclusion that the important cause of the longitudinal cracking that has developed in certain pavements of Kentucky, Indiana, and Illinois is the failure of the dummy longitudinal joint to crack through promptly as intended. In the 9–8–9 and 9–7–9-inch thickened-edge pavements of Kentucky and Indiana the delay in the fracturing of the center joints of this type was found mainly in the pavements that had been laid in the fall, but in the 10-inch uniform-thickness pavements of Illinois these joints have not always ruptured promptly even when the pavement was laid in the warm summer weather.

Consideration of the various conditions that tend to produce transverse tensile stress indicates that in a pavement that does not have complete partition of the concrete along the center line, the most important cause of transverse tensile stress in the region of the center line is restrained temperature warping. That this one

cause can produce stresses sufficiently great to break the full depth slab is evidenced by the many miles of center cracking in pavements built without a center joint 25 or 30 years ago and never subjected to heavy wheel loads. The studies of warping stresses conducted by the Public Roads Administration have furnished experimental evidence to the same effect.

Heavy wheel loads also produce high tensile stress in concrete slabs of usual thicknesses. However, except for loads applied at slab corners, the highest tensile stress is found directly under the wheels. Thus, even a heavy wheel load would not produce a high transverse tensile stress at the center line of the pavement unless the wheel were over the center line. With the vehicle in its normal position in one lane the transverse stress produced at the center line of the pavement would not be large. Any stress caused by the wheel load would combine with the warping stress that happened to be present, however.

In a pavement slab that has not broken along the center line restrained temperature warping causes a transverse tensile stress that reaches its maximum value at the center line of the pavement but has a magnitude almost as great for an appreciable distance on either side of the center line. Since this region of relatively high warping stress extends out into the vicinity of normal vehicle wheel travel, an unbroken dummy joint along the center line may result in the development of very high combined stresses in that part of the pavement where the heavy wheel loads are most frequent.

Partition of the slab along the center line greatly reduces the magnitude of the temperature warping stresses for given temperature conditions. The distribution of warping stress across the pavement is also changed and for a given wheel load the combined stress is greatly reduced.

This brief discussion of slab stresses has been introduced because it is believed that it offers a plausible explanation for the conditions that have been observed during this survey.

The pavements studied in this investigation, in which londitudinal cracking has occurred, have probably all been subjected to a greater amount of heavy traffic than they would have under normal conditions. It is probable also that had they not been subjected to this abnormally heavy traffic, the longitudinal joint would have broken as intended in a larger percentage of the panels before longitudinal cracking developed in the slabs themselves. While heavy traffic probably has been a contributing cause, these same traffic conditions have not caused longitudinal cracking in the parts of the 1942 pavements in which there was a definite separation at the longitudinal joint or in the older pavements of standard design in which there was also a definite separation at the longitudinal joint. This indicates that, even with traffic conditions as they have been, longitudinal cracking would not have occurred if the center joint had cracked as intended in all of the pavements.

It is indicated that the dummy longitudinal joint, as it was designed for the 1942 pavements, was defective in that it did not always develop promptly the plane of fracture as was intended. The remedy is a matter of design.

Past experience with the dummy longitudinal joint in highway pavements has generally been with slabs having a center depth of 8 inches or less. The usual practice has been to provide a groove 2 to 2½ inches in depth to create the weakened plane. The depth of the groove has been from one-fourth to one-third of the slab depth.

The 1942 pavements in Kentucky, Indiana, and Illinois were provided with a ribbon separation that extended nearly one-third of the pavement depth. The survey shows that these joints did not always fracture as was intended, which leads to the conclusion that a groove of this depth is not adequate.

The most logical remedy for longitudinal cracking of the type studied is the use of a longitudinal joint that will have definite separation at the time of construction. If the weakened-plane center joint is to be used it should be modified in some manner to give prompt fracture. Increasing the depth of the surface groove might be helpful. Were it not for the tie bars in the joint this groove could be carried to any desired depth. Since it is not desirable to have the bottom of the groove too close to the tie bars it may be necessary to place the tie bars at a slightly greater depth below the surface than is customary.

BEHAVIOR OF ASPHALTS IN THIN-FILM OVEN TEST

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by R. H. LEWIS, Senior Chemist, and W. J. HALSTEAD, Associate Chemist

IN APRIL 1941, a report was published by Lewis and Welborn on "The Properties of the Residues of 50–60 and 85–100 Penetration Asphalts from Oven Tests and Exposure." [1] This report presented the results of tests made on the residues from the standard oven test and from the thin-film oven test. The original asphalts were of the 50–60 and 85–100 grades. The thin-film oven test is the same as the standard oven test except that a 50-milliliter sample of asphalt is exposed in a flat-bottomed container 5.5 inches in diameter, the depth of the sample being approximately one-eighth inch.

The residues from the standard oven test were shown to be only slightly altered, and it was concluded that this test did not furnish adequate information concerning the probable behavior of asphalts for use in hot-mix paving. On the other hand, tests of the residues from the thin-film oven test showed that the asphalts were greatly altered by the test and, further, that the alterations were similar to those that have occurred in bitumens recovered from both laboratory and commercial paving-plant mixtures. It was concluded that this test would be useful in predicting the behavior of asphalts under processing and service conditions.

It was indicated that if an asphalt of the 50–60 grade produced a residue in the thin-film oven test ($\frac{1}{8}$-inch film, 5 hours heating at 325° F.) having 50 percent of the original penetration at 77° F. and a ductility over 40 centimeters at 77° F., there would be little danger of undue alteration occurring in the hot-mixing process and in service. To insure satisfactory performance by asphalts of the 85–100 penetration grade, requirements that the residue should have not less than 50 percent of the original penetration at 77° F. and a ductility of more than 100 centimeters at 77° F. were suggested.

The purpose of this study was to obtain similar information on representative samples of the 60–70, 100–120, and 120–150 penetration grades.

[1] PUBLIC ROADS, vol. 22, No. 2, April 1941. Also Proceedings of Association of Asphalt Paving Technologists, vol. 12, Dec. 1940.

STORED SAMPLES SHOWED CONSIDERABLE SURFACE HARDENING

The samples used for this study were from the same producers, and had been submitted at the same time as those tested and reported by Lewis and Welborn [1] in 1941 and in their earlier report on "The Physical and Chemical Properties of Petroleum Asphalts of the 50–60 and 85–100 Penetration Grades." [2] Samples were selected so as to include at least one sample from each general source of base petroleum, and also one sample of each asphalt from a source or producer that had supplied a sample of the 50–60 or 85–100 grade showing unusual behavior in the tests.

Seventeen asphalts from different sources or produced by different methods of manufacture were chosen. Data on the source and method of refining are shown in table 1. To facilitate comparisons, samples from the same source as samples used in the previous reports carry the same identification numbers. Samples corresponding to samples 20 and 29 of the 60–70 grade and samples 6 and 13 of the 120–150 grade were not available. A sample of the 40–50 grade of sample 29 was tested and the results listed with those for the 60–70 grade.

The samples had been stored in unopened metal containers for approximately 6 years and most of them gave evidence of surface hardening. It was believed that the surface material was oxidized or otherwise altered. To obtain a sample having more nearly the original characteristics, the top one-quarter inch of asphalt was removed from each can. The balance of the material was prepared for test by placing the container in an oven and heating at 230° F. until the asphalt was liquid enough to stir easily. The material was then stirred thoroughly.

Since the amount of hardening was of interest, the material cut from the top was tested for penetration at

[2] PUBLIC ROADS, March 1940, vol. 21, No. 1. Also Proceedings of Association of Asphalt Paving Technologist, vol. 11, January 1940.

TABLE 1.—*Source and method of refining asphalt cement*

Identification No.	Producer identification	Source of base petroleum	Method of refining
3	3	California, San Joaquin Valley field	Reduction and steam distillation.
6	6-A	Colombia	Vacuum distillation with pipe still.
8	8	Mexico	———
9	9	Mexico, Panuco field	Steam distillation in Trumble (pipe) still.
13	12	Venezuela	Fire and steam distillation.
14	13-A	Venezuela, Mene Grande field	Continuous distillation under subatmospheric pressure with steam.
15	13-B	do	Distilled in batch stills at atmospheric pressure with steam.
19	15	Arkansas, Smackover field	Vacuum distillation at a low temperature.
20	16-A	do	Vacuum distillation, 89 m. p. flux.
29	21	Oklahoma, Healdton and Graham	———
30	22	Kentucky and Illinois	Fire and steam distillation, possibly blown.
32	24-A	Kansas	Straight run, steam refined, vacuum process.
33	24-B	do	Produced from Winkler-Koch Shell still.
34	25-A	Wyoming	Fire and steam distillation.
35	25-B	Unknown	do
37	26-A	Mexico and Domestic Gulf Coast	———
40	28	Kentucky	Dubbs cracking process.

TABLE 2.—*The surface hardening of asphalts in cans after aging approximately 6 years*

Identification No.	Type of metal container	Penetration at 77° F., 100 gm., 5 sec.								
		60–70 grade			100–120 grade			120–150 grade		
		Top [1]	Total [2]	Relation top to total	Top	Total	Relation top to total	Top	Total	Relation top to total
				Percent			*Percent*			*Percent*
3	[3]	34	67	51	59	103	57	100	138	72
6	Slip top, cylindrical	33	66	50	59	116	51			
8	Double friction top, cylindrical	32	65	49	53	107	50	60	130	46
9	Single friction top, cylindrical	80	80	100	102	108	94	131	132	99
13	Double friction top, cylindrical	38	60	63	55	109	50			
14	do	27	65	41	85	117	73	131	136	96
15	do	29	61	47	50	105	48	111	121	90
19	Small screw cap, rectangular	60	63	95	92	101	91	127	126	101
20	Single friction top, cylindrical				82	99	83	117	129	91
29	Small screw cap, rectangular	[4] 20	[4] 45	[4] 44	107	108	99	189	164	115
30	Single friction top, cylindrical	35	65	54	79	107	74	121	131	92
32	Clamped top, bucket type	32	62	52	53	105	50	63	121	52
33	do	32	57	56	49	97	51	54	110	49
34	Large screw cap, rectangular	34	66	52	62	102	61	88	123	72
35	Double friction top, cylindrical	37	60	62	101	117	86	115	135	85
37	Clamped top, bucket type	40	61	66	65	115	57	74	120	62
40	Small screw cap, rectangular	30	64	47	52	106	49	70	132	53

[1] Material cut from approximately the top one-fourth inch of sample.
[2] Material for test taken after removal of top one-fourth inch and sample had been heated and stirred thoroughly.
[3] 60–70 material in large screw-cap, rectangular container; 100–120 material in single friction top, cylindrical container; 120–150 material in small screw-cap, rectangular container.
[4] These values are for 40–50 grade.

TABLE 3.—*Test characteristics of the 60–70 penetration grade asphalts and their residues from the thin-film oven test*

Identification No.	Original asphalt									Thin-film oven test (⅛-in. film, 5 hr. at 325° F.)							
	Oliensis spot test	Specific gravity at 77° F.	Softening point	Penetration, 100 gm. 5 sec. at—			Ductility, 5 cm. per min. at—			Change in weight	Tests on the residue						
											Softening point	Penetration, 100 gm. 5 sec. at—			Ductility, 5 cm. per min. at—		
				50° F.	60° F.	77° F.	50° F.	60° F.	77° F.			50° F.	60° F.	77° F.	50° F.	60° F.	77° F.
			°F.				*Cm.*	*Cm.*	*Cm.*	*Percent*	°F.				*Cm.*	*Cm.*	*Cm.*
3	Negative	1.013	119	9	19	67	195	250+	250+	−0.08	126	6	13	41	[1] 0	250+	250+
6	do	1.006	126	14	24	66	8	43	219	+.06	139	11	18	41	4.5	9	114
8	Positive	1.041	131	20	30	65	5.5	11.5	42	−.26	148	13	19	36	3	4	7.5
9	Negative	1.039	126	20	33	80	32	201	250+	−.65	139	14	22	46	6	16.5	108
13	Positive	1.025	132	17	28	60	6.5	15	146	−.19	151	12	17	32	3.5	4.5	9
14	do [2]	1.024	124	13	23	65	8.5	142	250+	+.17	133	10	17	42	5.5	12	206
15	do	1.034	127	13	22	61	9	41	250+	−.004	141	10	15	35	4	6.5	45
19	Negative	1.024	124	13	23	63	6.5	57	236	+.11	133	10	18	43	5.5	10	187
29 [3]	Positive	1.039	126	9	15	45	4.5	17	250+	−.07	144	6	11	24	[1] .5	3.5	10
30	Negative	1.005	127	15	25	65	7.5	23.5	193	+.08	140	12	20	44	4.5	7	38
32	do	1.012	129	13	24	62	6	25	166	+.11	140	10	17	40	4.5	7	34
33	Positive	1.034	128	14	23	57	4.3	8.5	36	+.05	151	11	19	39	3	4	6
34	do	1.037	128	18	29	66	6.5	16.5	180	+.07	146	14	20	36	3.5	4.5	12
35	Negative	1.011	131	18	27	60	5.3	11.5	167	−.09	139	14	20	40	4	6	22
37	do	1.011	130	19	29	61	8	24	144	−.29	146	14	20	40	4.5	6	31
40	Positive	1.052	123	13	22	64	8.8	57	250+	−2.10	148	7	10	22	[1] 0	3	8

[1] Specimen broke. [2] Very slight spot. [3] 40–50 grade; no 60–70 grade available.

77° F. (100 grams, 5 seconds). Comparison of the penetration of this top material with that of the stirred sample, which will be referred to as the total sample, is shown in table 2. This table also shows the type of metal container in which each material had been stored.

A majority of the samples showed considerable hardening at the surface. The exceptions were all grades of samples 9 and 19; the 100–120 and 120–150 grades of sample 29, and the 120–150 grades of samples 14, 15, 20, and 30. In all these samples the penetration of the top material was equal to 90 percent or more of the penetration of the total sample. Sample 29 of the 120–150 grade was unusual in that the top material was softer than the total sample. When opened, the surface of this sample had drops of water on it, even though the material was in a container with a small, well-sealed opening. Considerable difficulty was experienced from foaming when heating this material.

There was no definite trend in the amount of hardening. For example, in the case of sample 14, the top material of the 60–70 grade had a penetration equal to 41 percent of the penetration of the total sample. For the 100–120 grade the penetration of the top material was 73 percent of that of the total sample and there was practically no difference for the 120–150 grade, the top having a penetration of 96 percent of that of the total sample. All of these samples were in similar containers and approximately the same amount of material was in each can. Sample 29, which was in cans having a small screw cap, also showed variation in penetration ratios to a marked degree. The top of the 40–50 grade had a penetration of 44 percent of that of the total sample; in the 100–120 grade it had 99 percent; and in the 120–150 grade, 115 percent.

However, despite the surface hardening of these samples there is no indication that the bulk of the material had changed appreciably since, in most cases, the penetration was still within the required limits for

TABLE 4.—*Test characteristics of the 100–120 penetration grade asphalts and their residues from the thin-film oven tests*

Identification No.	Original asphalt: Oliensis spot test	Specific gravity at 77° F.	Softening point	Penetration, 100 gm. 5 sec. at —			Ductility, 5 cm. per min. at —			Thin-film oven test (⅛-in. film, 5 hr. at 325° F.): Change in weight	Tests on the residue: Softening point	Penetration, 100 gm. 5 sec. at —			Ductility, 5 cm. per min. at —		
				50° F.	60° F.	77° F.	50° F.	60° F.	77° F.			50° F.	60° F.	77° F.	50° F.	60° F.	77° F.
			°F.				Cm.	Cm.	Cm.	Percent	°F.				Cm.	Cm.	Cm.
3	Negative	1.011	114	14	29	103	250+	250+	175	−0.20	119	9	17	61	62	250+	250+
6	do	1.002	113	22	41	116	185	250+	152	+.03	122	18	29	70	11.5	84	185
8	Positive	1.036	119	30	47	107	12.5	45	98	−.50	144	15	23	47	3.5	5	13
9	Negative	1.036	120	27	46	108	105	250+	221	−.88	135	15	24	56	7.5	20	115
13	Positive	1.027	119	29	47	109	36	88	215	−.33	137	17	25	49	5.5	8	31
14	do [1]	1.018	115	23	41	117	64	250+	225	+.11	123	15	26	67	10.5	62	180
15	do	1.027	117	23	39	105	66	250+	152	+.01	128	14	23	53	5.5	21	194
19	Negative	1.019	115	21	37	101	23	250+	144	+.12	125	16	27	65	8	43	230
20	do	1.013	121	32	49	99	11.5	46	165	−.77	135	22	32	62	5	8	60
29	Positive	1.026	114	16	33	108	66	250+	140	−.04	127	10	18	46	5.5	13	250+
30	Negative	.982	119	30	46	107	15	66	106	+.07	126	24	37	80	8	14.5	98
32	do	1.007	116	21	37	105	15.5	140	158	+.15	129	17	27	62	6.5	15.5	149
33	Positive	1.028	119	20	35	97	10	14	85	+.01	138	16	25	57	5	6	9.5
34	do	1.031	117	24	40	102	31	130	160	−.17	131	15	23	51	4.5	10	60
35	Negative	1.006	112	26	45	117	25	125	124	−.17	121	20	33	78	9	19.5	160
37	do	1.010	118	33	52	115	17	144	166	−1.60	139	18	27	53	4.5	7.5	35
40	Positive	1.049	111	20	37	106	57	250+	165	−2.38	140	8	13	29	[2] 1.5	4.8	18

[1] Very slight spot. [2] Specimen broke.

TABLE 5.—*Test characteristics of the 120–150 penetration grade asphalts and their residues from the thin-film oven test*

Identification No.	Original asphalt: Oliensis spot test	Specific gravity at 77° F.	Softening point	Penetration, 100 gm. 5 sec. at —			Ductility, 5 cm. per min. at —			Thin-film oven test (⅛-in. film, 5 hr. at 325° F.): Change in weight	Tests on the residue: Softening point	Penetration, 100 gm. 5 sec. at —			Ductility, 5 cm. per min. at —		
				50° F.	60° F.	77° F.	50° F.	60° F.	77° F.			50° F.	60° F.	77° F.	50° F.	60° F.	77° F.
			°F.				Cm.	Cm.	Cm.	Percent	°F.				Cm.	Cm.	Cm.
3	Negative	1.008	108	19	41	138	250+	250+	142	−0.34	114	14	26	84	250+	250+	147
8	Positive	1.033	114	31	53	130	30	113	92	−.51	138	20	29	57	5	7.5	23
9	Negative	1.033	113	33	55	132	183	225	161	−.78	132	20	33	70	12	47.5	88
14	Positive [1]	1.016	109	25	48	136	240	250+	161	+.11	121	18	32	79	15	104	167
15	do	1.026	111	24	46	121	134	250+	132	−.04	126	17	28	62	10	31.5	167
19	Negative	1.017	111	24	44	126	62	229	110	+.11	119	18	33	80	1.5	88	139
20	do	1.012	112	35	58	129	34	155	123	−.81	126	24	38	77	7	19.5	95
29	Positive	1.022	105	23	46	164	190+	230	120	−.52	124	15	26	65	7	22.5	196
30	Negative	.983	114	32	54	131	32	133	116	+.04	121	27	45	95	11.5	42	88
32	do	1.005	112	24	43	121	68	168	130	+.12	123	20	30	72	10	28.5	93
33	Positive	1.029	112	23	42	110	14	33	81	−.10	134	18	30	65	4.5	6	10.5
34	do [1]	1.023	112	24	43	123	107	195	107	+.13	124	18	31	72	14	115	145
35	Negative	1.005	110	31	53	135	75	250+	115	−.27	118	23	38	88	12	68	157
37	do	1.004	118	40	62	120	15	32	79	−.72	140	29	39	67	5	6.5	15
40	Positive	1.048	107	25	47	132	250+	250+	105	−2.44	136	11	18	34	3.5	6.5	32

[1] Very slight spot.

the designated grades. The exceptions are sample 9 of the 60–70 grade, which had a penetration of 80; sample 20 of the 100–120 grade, which had a penetration or 99; sample 29 of the 120–150 grade, which had a penetration of 164; and all grades of sample 33, their values being 57 for the 60–70 grade, 97 for the 100–120 grade, and 110 for the 120–150 penetration grade. The results for these particular samples have been included under the original grade designations since in most cases the deviations from the required penetration were small.

The previous reports have indicated that the changes in the physical properties of asphalts as measured by the consistency and ductility are of a greater magnitude and thus of more significance than chemical changes as exemplified by solubility tests. It has also been shown that the standard oven-loss test is of little value in predicting the durability of the asphalt. Therefore, in this study only the Oliensis spot test, specific gravity at 77° F., softening point, penetration (100 grams, 5 seconds) at 50°, 60°, and 77° F., and ductility (5 centimeters per minute) at the same three temperatures were determined for the original asphalt. The thin-film oven test (one-eighth inch, 5 hours, at 325° F.) was made and the residue was tested for softening point, penetration at 50°, 60°, and 77° F., and ductility at these three temperatures. Table 3 shows the results of these tests for the 60–70 penetration grade, table 4 shows the results for the 100–120 grade, and table 5 shows the results for the 120–150 grade.

A summary of the changes in weight and the changes in penetration and softening point of the residues expressed as percentages of the original test values, are shown in table 6. This table also includes data for the 50–60 and 85–100 penetration grades which were taken from the earlier report by Lewis and Welborn. There seems to be little significance in the amount of change in weight of the samples, some show a slight decrease while others show an increase. Only six samples had losses in weight greater than 1 percent, these being all grades of sample 40, and the 100–120 grade of sample 37. In addition to these samples there were only one sample of the 60–70 grade, two samples of the 85–100 grade, two of the 100–120 grade, and five of the 120–150 grade which had a loss greater

TABLE 6.—*Changes in test characteristics for various penetration grades of asphalt after the thin-film oven test*

Identification No.	Penetration grade														
	50–60			60–70			85–100			100–120			120–150		
	Change in weight	Relation of test value of residue to original		Change in weight	Relation of test value of residue to original		Change in weight	Relation of test value of residue to original		Change in weight	Relation of test value of residue to original		Change in weight	Relation of test value of residue to original	
		Penetration at 77° F., 100 gm. 5 sec.	Softening point		Penetration at 77° F., 100 gm. 5 sec.	Softening point		Penetration at 77° F., 100 gm. 5 sec.	Softening point		Penetration at 77° F., 100 gm. 5 sec.	Softening point		Penetration at 77° F., 100 gm. 5 sec.	Softening point
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
3	+0.01	64	106	−0.08	61	106	−0.14	59	106	−0.20	59	104	−0.34	61	105
6	0.	67	109	+.06	62	110	+.04	63	106	+.03	60	108			
8	−.22	61	117	−.26	55	113	−.46	45	119	−.50	44	121	−.51	44	121
9	−.20	66	110	−.65	58	110	−.55	53	112	−.88	52	112	−.78	53	117
13	−.24	61	115	−.19	53	114	−.11	50	114	−.33	45	115			
14	+.09	63	107	+.17	65	107	+.05	59	113	+.11	57	107	+.11	58	111
15	0.	52	111	−.004	57	111	+.01	57	115	+.01	50	109	−.04	51	113
19	+.10	68	106	+.11	68	107	+.09	68	108	+.12	64	109	+.11	64	107
20	−.46	67	107				−.59	68	109	−.77	63	112	−.81	59	112
29 [1]				−.07	53	114	−.13	43	114	−.04	43	111	−.52	40	118
30	+.05	71	105	+.08	68	110	+.08	69	111	+.07	75	106	+.04	73	106
32	+.08	67	107	+.11	64	109	+.09	62	111	+.15	59	111	+.12	60	110
33	+.05	72	120	+.05	68	118	+.06	60	117	+.01	59	116	−.10	59	120
34	+.04	55	116	+.07	54	114	+.04	52	115	−.17	50	112	+.13	59	111
35	−.10	72	105	−.09	67	106	−.14	68	107	−.17	67	108	−.27	65	107
37	−.13	60	111	−.27	66	112	−.12	58	114	−1.60	46	118	−.72	56	119
40	−2.09	38	124	−2.10	34	120	−2.17	32	123	−2.38	27	126	−2.44	26	127

[1] Values listed under the 60–70 grade are for the 40–50 grade.

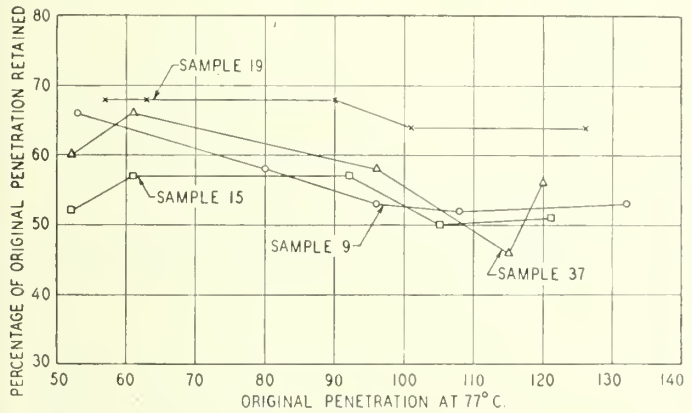

FIGURE 1.—PERCENTAGE OF PENETRATION RETAINED COMPARED TO THE ORIGINAL PENETRATION AT 77° C.

than 0.5 percent. Most specifications for the standard oven-loss test allow a maximum loss of 1 percent, and these results indicate that if the thin-film test were used in specifications, this same value would be suitable for all the grades up to and including the 120–150 penetration grade.

REQUIREMENT THAT RESIDUE HAVE 50 PERCENT OF ORIGINAL PENETRATION IS NOT UNDULY SEVERE

The percentage of original penetration retained by the residue from the thin-film oven test shows a general tendency to decrease with increases in original penetration but there are numerous deviations from this trend. In a number of cases the percentage of penetration retained by an asphalt of the 120–150 grade is greater than that retained by the corresponding asphalt of the 100–120 grade. Typical curves showing the relation between original penetration and percentage of retained penetration are shown in figure 1. As would be expected, the percentage increase in softening point of the residue, as compared with that of the original material, shows a general tendency to increase as the percentage of retained penetration decreases. However, no significant relation between increase in softening point and other properties of the materials could be established.

As previously stated, it has been suggested that, for asphalts of the 50–60 and 85–100 penetration grades, the residue from the thin-film oven test be required to have a penetration not less than 50 percent of the original penetration. Table 6 shows that such a requirement would not be unduly severe if applied to all grades between 50 and 150 penetration. The samples which fail to meet such a requirement are the 85–100, 100–120, and 120–150 grades of sample 8; the 100–120 grade of sample 13; the 85–100, 100–120, and 120–150 grades of sample 29; the 100–120 grade of sample 37, and all grades of sample 40.

WHEN COMPARED AT THE SAME PENETRATION, RESIDUE FROM THIN-FILM OVEN TEST HAS LOWER DUCTILITY THAN THE ORIGINAL ASPHALT

The ductilities of the residues from the thin-film oven test compared to those of the original asphalts at the same temperature fails to give a clear picture of the amount of change caused by the heating, since the change in the consistency may either decrease or increase the ductility depending on the original penetration.

In order to obtain a better basis for comparison, the double logarithm of the ductility was plotted against the penetration at which the ductility was obtained, the values for all grades and at all test temperatures being plotted on the same graph. It was found in most cases that the values for the asphalts, before the oven-heat test, form a smooth curve with the maximum ductility occurring at a penetration which is usually between 50 and 90 for the asphalts tested in this investigation. The data for the residues from the oven-heat test form a second curve which is below that for the original asphalts. The exceptions to this behavior are samples 8, 20, 33, 34, and 37 which have separate curves for each penetration grade. A typical curve similar to those obtained for a majority of the

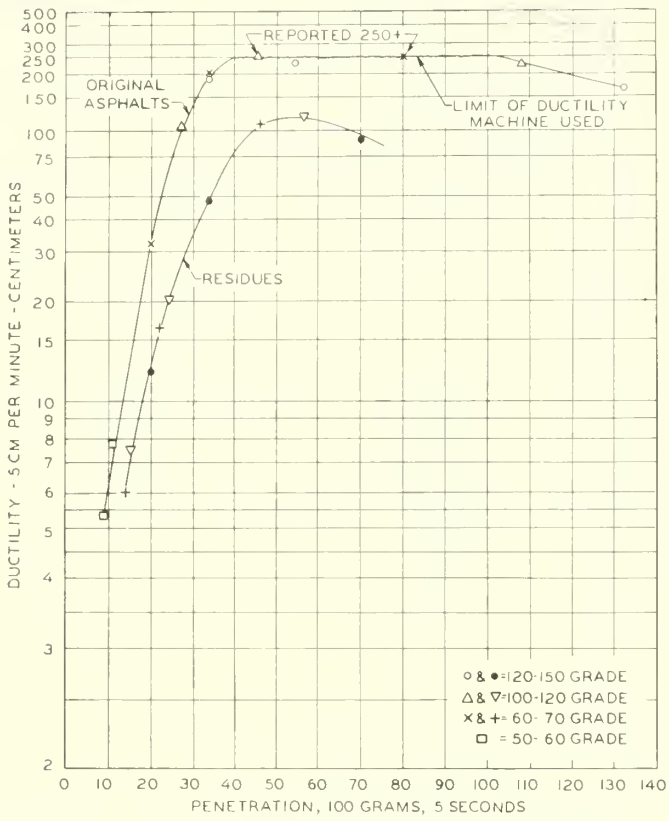
REPORTED 250+
ORIGINAL ASPHALTS
LIMIT OF DUCTILITY MACHINE USED
RESIDUES
○ & ● = 120-150 GRADE
△ & ▽ = 100-120 GRADE
× & + = 60-70 GRADE
□ = 50-60 GRADE
DUCTILITY - 5 CM. PER MINUTE - CENTIMETERS
PENETRATION, 100 GRAMS, 5 SECONDS

FIGURE 2.—RELATION BETWEEN DUCTILITY AND PENETRATION FOR SAMPLE 9.

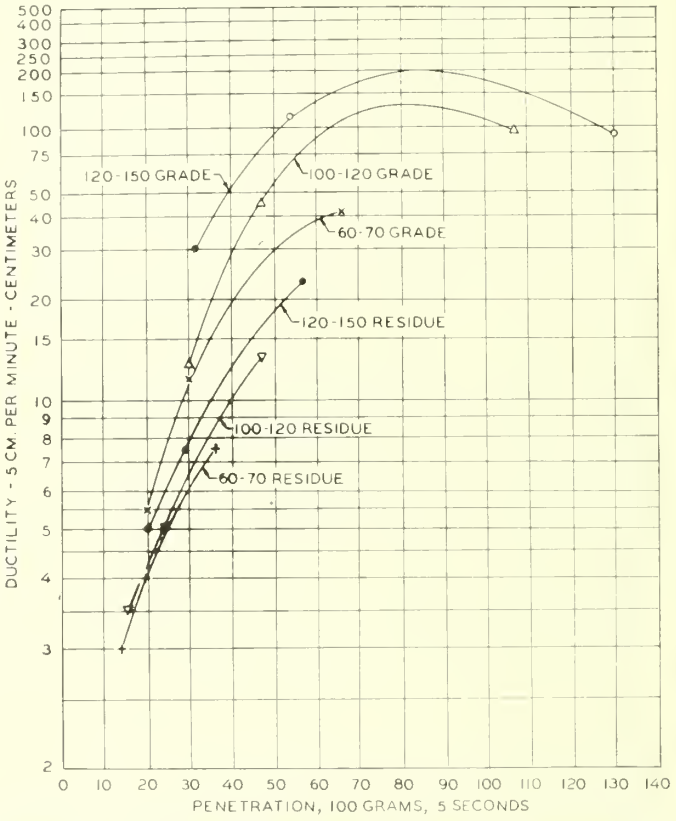

FIGURE 3.—RELATION BETWEEN DUCTILITY AND PENETRATION FOR SAMPLE 8.

samples and their residues is shown in figure 2, while figure 3 exemplifies the wide variation obtained with a few asphalts. It is interesting to note that both sample 9 of figure 2 and sample 8 of figure 3, were manufactured from Mexican petroleum, but sample 9 was steam distilled while sample 8 was refined by some process which gave a residue with a positive spot when tested by the Oliensis method.

From the curves constructed as indicated a close approximation of the ductility of each original asphalt at any penetration from approximately 10 to 130 can be made. Table 7 shows the penetration and ductility of all the residues and the ductility of the original asphalt (estimated from the curve) at the penetration of the residue. When the comparison is made on this basis it is seen that the ductility of the residue is always equal to or less than that of the original sample.

There is only a small difference between the ductilities for the lower penetrations, which approach very closely the lower limit of measurable ductility, but as the penetration becomes greater the difference increases. At 77° F., excepting the samples which have a ductility of the original material greater than 250 centimeters which makes the percentage indeterminable, only four samples have residues with a ductility equal to more than 50 percent of that of the original at the penetration of the residue, and in most cases the proportion of the ductility destroyed is much greater than this. These samples are the 100–120 grade of 20, the 100–120 grade of 32, the 120–150 grade of 34, and the 60–70 grade of 37. These results show that, in addition to the change in ductility caused by the change in consistency of the samples when they are heated in the thin-film oven test, there is an additional decrease in the ductility caused by the heating and the amount of this decrease varies considerably for the samples from various sources and methods of manufacture.

SUGGESTED REQUIREMENTS GIVE COMPARABLE RESULTS FOR ALL GRADES

Comparison of ductilities at the same penetration as shown in table 7 is impractical as a specification requirement, the actual measurement of the ductility at 77° F. being a more simple means of control. Lewis and Welborn [3] suggested ductility requirements of not less than 40 centimeters at 77° F. (5 centimeters per minute) for the 50–60 grade and not less than 100 centimeters at 77° F. for the 85–100 grade. Using these figures as the basis, the results indicate that reasonable requirements for the other grades tested would be: Not less than 40 centimeters at 77° F. (5 centimeters per minute) for the 60–70 grade and not less than 100 centimeters at 77° F. for the 100–120 and 120–150 grades. Table 8 is a summary of the asphalts of all the grades tested, indicating those that would fail to meet the proposed requirements for penetration and ductility. In general, those samples which fail in one grade fail in all grades. In all cases except the 100–120 and 120–150 grades of sample 29, the samples which fail to retain at least 50 percent of the original penetration at 77° F. also fail to meet the ductility requirement.

It has been shown in a previous report [4] that if the logarithm of the penetration is plotted against the temperature in degrees Fahrenheit a straight line will

[3] See footnote 1.
[4] See footnote 2.

TABLE 7.—*The effect of the thin-film oven test on the ductility-penetration relationship*

Identification No.	Temperature of test	Penetration grade— 60–70: Tests on residue: Penetration, 100 gm., 5 sec.	60–70: Tests on residue: Ductility, 5 cm. per min.	60–70: Approximate ductility of original asphalt at equal penetration	100–120: Tests on residue: Penetration, 100 gm., 5 sec.	100–120: Tests on residue: Ductility, 5 cm. per min.	100–120: Approximate ductility of original asphalt at equal penetration	120–150: Tests on residue: Penetration, 100 gm., 5 sec.	120–150: Tests on residue: Ductility, 5 cm. per min.	120–150: Approximate ductility of original asphalt at equal penetration
	°F.		*Cm.*	*Cm.*		*Cm.*	*Cm.*		*Cm.*	*Cm.*
3	77	41	250+	250+	61	250+	250+	84	147	250+
	60	13	250+	250+	17	250+	250+	26	250+	250+
	50	6	[1] 0	[2]	9	62	[2]	14	250+	250+
6	77	41	114	250+	70	185	250+			
	60	18	9	28	29	84	220			
	50	11	4.5	6	18	11.5	28			
8	77	36	7.5	15	47	13	45	57	23	120
	60	19	4	5.5	23	5	8.5	29	7.5	24
	50	13	3	3	15	3.5	3.5	20	5	10
9	77	46	108	250+	56	115	250+	70	88	250+
	60	22	16.5	50	24	20	70	33	47.5	190
	50	14	6	10.5	15	7.5	13	20	12	32
13	77	32	9	29	49	31	100			
	60	17	4.5	6.5	25	8	17			
	50	12	3.5	4	17	5.5	6.5			
14	77	42	206	250+	67	180	250+	79	107	250+
	60	17	12	22	26	62	220	32	104	250+
	50	10	5.5	5.5	15	10.5	15	18	15	36
15	77	35	45	250+	53	194	250+	62	167	250+
	60	15	6.5	12	23	21	90	28	31.5	250+
	50	10	4	5	14	5.5	9.5	17	10	15
19	77	43	187	250+	65	230	250+	80	139	250+
	60	18	10	10	27	43	120	33	88	250+
	50	10	5.5	5.5	16	8	8	18	11.5	11.5
20	77				62	60	80	77	95	200
	60				32	8	18	38	19.5	43
	50				22	5	5	24	7	12
29 [3]	77	24	10	250+	46	250+	250+	65	196	250+
	60	11	3.5	6.5	18	13	200	26	22.5	250+
	50	6	[1] .5	[2]	10	5.5	5.5	15	7	30
30	77	44	38	80	80	98	215	95	88	210
	60	20	7	17	37	14.5	46	45	42	85
	50	12	4.5	6	24	8	17	27	11.5	21
32	77	40	34	160	62	149	220	72	93	220
	60	17	7	7.5	27	15.5	75	30	28.5	100
	50	10	4.5	4.5	17	6.5	7.5	20	10	12
33	77	39	6	38	57	9.5	72	65	10.5	90
	60	19	4	9.5	25	6	15	30	6	23
	50	11	3	4	16	5	7	18	4.5	9
34	77	36	12	36	51	60	180	72	145	200
	60	20	4.5	7.5	23	10	31	31	115	160
	50	14	3.5	4	15	4.5	6	18	14	110
35	77	40	22	170	78	160	250+	88	157	250+
	60	20	6	6	33	19.5	95	38	68	150
	50	14	4	4	20	9	9	23	12	12
37	77	40	31	60	53	35	125	67	15	40
	60	20	6	8.5	27	7.5	20	39	6.5	15
	50	14	4.5	6.5	18	4.5	8	29	5	12
40	77	22	8	57	29	18	250+	34	15	250+
	60	10	3	5	13	4.7	8.7	18	6.5	30
	50	7	[1] 0	3	8	[1] 1.5	4	11	3.5	5.5

[1] Specimen broke.
[2] Could not be estimated.
[3] Values listed under 60–70 grade are for 40–50 penetration grade.

connect the points. The slope of this line is a measure of the susceptibility of the material to changes in consistency with temperature changes.

These slopes may be determined from the graph or calculated from any two values of the penetration and the temperatures at which those penetrations occur with the following equation:

$$\text{slope} = \frac{\log p_2 - \log p_1}{t_2 - t_1}$$

where p_2 and p_1 are the penetrations (100 grams, 5 seconds) at the two temperatures, t_2 and t_1, respectively.

Table 9 shows these slopes for the asphalts tested for the five penetration grades from 50–60 through 120–150. The slopes for the 60–70, 100–120, and 120–150 grades were calculated by the equation using the penetrations at 77° F. and at 50° F. given in tables 3, 4, and 5. The slopes for the 50–60 and 85–100 grades were taken from an earlier report.[5] A difference of one point in the pene-

[5] See footnote 2.

TABLE 8.—*Identification of the asphalts which fail to meet the proposed requirements for the thin-film oven test* [1]

Identification No.	50–60 penetration		60–70 penetration		85–100 penetration		100–120 penetration		120–150 penetration	
	Proposed requirement									
	Penetration at 77° F. 50+ percent	Ductility at 77° F. 40+ cm.	Penetration at 77° F. 50+ percent	Ductility at 77° F. 40+ cm.	Penetration at 77° F. 50+ percent	Ductility at 77° F. 100+ cm.	Penetration at 70° F. 50+ percent	Ductility at 77° F. 100+ cm.	Penetration at 77° F. 50+ percent	Ductility at 77° F. 100+ cm.
3									N	N
6		O		O	X	X	X	O	X	O
8										X
9		X		X		X	X	X	N	N
13										
14		X								
15										
19		O	N	N		X		X		X
20	N	N		X	X	X	X		X	
29 [2]		X		X				X		X
30		X		X		X				X
32		O		O		X		O		O
33		X		X		X		X		
34				X						
35		X		X		X	X	X		O
37	X	X	X	X	X	X	X	X	X	X
40										

[1] Symbols used are as follows:
X denotes the samples which fail to meet the requirement; O denotes those samples which fail to meet the requirement and also had an initial ductility of less than 100 cm. at 77° F.; and N denotes no test.
[2] Data listed under 60–70 grade are for 40–50 grade.

TABLE 9.—*The effect of the thin-film oven test on the susceptibility to changes in consistency with changes in temperature*

Identification No.	Slope of log penetration-temperature curve for—							
	50–60 grade original	60–70 grade		85–100 grade original	100–120 grade		120–150 grade	
		Original	Residue		Original	Residue	Original	Residue
3	30×10^{-3}	32×10^{-3}	31×10^{-3}	32×10^{-3}	32×10^{-3}	31×10^{-3}	32×10^{-3}	29×10^{-3}
6	25	25	21	27	27	22		
8	19	19	16	21	20	18	23	17
9	23	22	19	22	22	21	22	20
13	19	20	16	22	21	17		
14	24	26	23	25	26	24	27	24
15	22	25	20	25	24	21	26	21
19	25	25	23	24	25	23	27	24
20	17			19	18	17	21	19
29 [1]		26	22	27	31	25	32	24
30	20	24	21	25	20	19	23	20
32	23	24	22	23	26	21	26	21
33	23	23	20	22	25	20	25	21
34	22	21	15	23	23	19	26	22
35	23	19	17	22	24	22	24	22
37	19	19	17	20	20	17	18	14
40	24	26	18	26	27	21	27	18

[1] Values listed under the 60–70 grade are for the 40–50 penetration grade.

tration at 50° F. changes the value of the slope in the second significant figure and thus, since this is within the experimental error, only two significant figures are reported. It is seen from table 9 that in every case the residue from the thin-film oven test has a lower slope and thus has become less susceptible to temperature changes than the original asphalt. This result indicates that some oxidization takes place during the test.

There is no definite relation between the susceptibilities of the original asphalts and the ductilities or penetrations of the residues from the thin-film oven test except that the asphalts of low susceptibility consistently fail to meet the proposed ductility requirements. However, the converse is not true since the residues from a number of the asphalts of high susceptibility also fail to pass the proposed ductility requirements. Table 9 gives the slopes of the log penetration-temperature curves for 81 samples of original asphalt. The slopes for 20 of these samples are 21×10^{-3} or less and the residues from all these materials failed to meet the ductility requirements. The remaining 61 samples of higher susceptibility are divided about equally between negative-spot and positive-spot materials, there being 32 of the former and 29 of the latter. The residues from 7 of the 32 negative-spot materials and 18 of the 29 positive-spot materials failed to meet the ductility requirements. The residues from 2 of the 11 positive-spot materials that met the ductility requirements failed to meet the proposed penetration requirements.

CONCLUSIONS

The results of the tests described in this report justify the following conclusions:

1. For asphalts manufactured by the same refining process from the same base petroleum the relation of the ductility to penetration is the same for all penetration grades regardless of the temperature at which a given penetration occurs.

2. The ductilities of the residues from the thin-film oven test show a common relation to penetration that is different from the relation for the original asphalt, the ductility of the residue being generally less than that of the original sample when the comparison is made at the same penetration.

3. A requirement that the sample shall have a loss in weight not greater than 1 percent when heated in an ⅛-inch film for 5 hours at 325° F. appears to be suitable for all penetration grades of asphalt from the 50–60 through the 120–150 grades.

4. A requirement that the residue from the thin-film oven test (⅛-inch film heated 5 hours at 325° F.) shall have a penetration at 77° F. (100 gram, 5 seconds) of at least 50 percent of that of the original sample appears to be equally suitable for all penetration grades of asphalt from the 50–60 through the 120–150 grades.

5. The requirement that the residue from the thin-film oven test (⅛-inch film heated 5 hours at 325° F.) shall have a ductility at 77° F. (5 centimeters per minute) of not less than 40 centimeters for the 50–60 and 60–70 penetration grades, and not less than 100 centimeters for the 85–100, 100–120, and 120–150 penetration grades gives comparable results for all these grades.

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A STUDY OF BITUMINOUS-COATED CORRUGATED SHEET METAL CULVERTS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by J. Y. Welborn, Associate Highway Engineer, and P. J. Serafin, Assistant Chemical Engineer

IN THE CONSTRUCTION of certain sections on the Blue Ridge parkway during 1936–41, bituminous-coated corrugated sheet metal culverts were installed for the drainage of surface water. Preliminary inspection of these pipe was made by United States naval inspectors at the plants of the fabricators, at which time the erosion test was made. Samples of the pipe were then sent to the Public Roads laboratory, where the asphalt coating and galvanized metal were tested for conformity with the governing specifications.

Since these culvert pipe have been in service from 4 to 9 years it seemed advisable to determine their service behavior. This report covers the field inspection of the pipe in the fall of 1945 and laboratory analyses of the samples taken.

The types of bituminous-coated metal culverts and the sections of road on which they are used are given in table 1.

In order that the reader may distinguish between the types of culvert pipe used on the various sections and have a better understanding of the conditions found during the inspection, brief descriptions of the three general types of bituminous-coated sheet metal pipe are given below.

TABLE 1.—*Types of culvert pipe installed on the Blue Ridge parkway*

Type of culvert pipe	Section	Fabricator	Number of pipe culverts installed	Number of culverts inspected	Date of construction
Galvanized metal coated with asphalt and invert paved with asphalt	2M–1	A	79	26	1938–39
	2U–2	B	67	11	1939–40
Asbestos-bonded metal coated with asphalt and invert paved with asphalt	2M–2	A	38	11	1936–38
	2N–1	A	87	26	1936–38
	2P–2	A	110	31	1937–38
Galvanized metal coated with asphalt and invert paved with metal reinforced asphalt	2N–2	C	69	17	1940–41
	2P–1	C	(1)	7	(1)
	2V–1	C	45	11	1939–40

[1] Section not completed. Construction started in 1940, stopped in 1941.

TYPES OF BITUMINOUS-COATED CORRUGATED SHEET METAL CULVERT PIPE DESCRIBED

Galvanized metal pipe coated with asphalt and with an asphalt pavement.—Galvanized corrugated sheet metal pipe is uniformly coated with asphalt on the outside and the inside. One-fourth of the inside circumference, which forms the bottom of the pipe when installed, is coated with an additional thickness of asphalt such that the corrugations in the pipe are filled and a smooth pavement formed. The specifications required that the thickness of asphalt coating on the inside three-fourths of the circumference should be not less than three-hundredths inch and on the bottom quarter that the thickness should be sufficient to meet the requirements of the erosion test.

Asbestos-bonded metal pipe coated with asphalt and with an asphalt pavement.—In the process of galvanizing the metal sheet for use in the fabrication of this type of culvert pipe a layer of asbestos felt is rolled into the molten spelter coating on one or both sides of the sheet. In the pipe covered by this report the asbestos felt was on only one side of the sheet. On cooling, a portion of the asbestos felt becomes securely embedded in the coating, leaving a mass of fiber on the surface of the sheet. This surface is then primed with a bituminous material after which the sheets are fabricated into corrugated pipe. The asbestos-bonded surface formed the inside of the pipe inspected. As a final step in manufacture the pipe is coated with asphalt and the asphalt pavement formed as for the plain galvanized metal pipe.

It is claimed that the asbestos treatment assures permanent tight adhesion of the asphalt coating, increases the resistance of the asphalt coating to fracture on impact, and tends to cushion pressure caused by water freezing within the pipe.

Galvanized metal pipe coated with asphalt and with a metal reinforced asphalt pavement.—Galvanized corrugated sheet metal is used in the fabrication of this kind of pipe. The outside of the pipe and three-fourths of the inside circumference are uniformly coated with asphalt but the remaining fourth, which forms the bottom of the pipe when installed, is paved with asphalt reinforced with metal.

In fabricating the pavement a layer of asphalt-impregnated felt is placed inside the pipe over the corrugations and over this a sheet of metal, curved to fit the circumference of the pipe, is placed. The felt and the metal plate cover approximately one-fourth of the circumference of the pipe and are the same length as each corrugated section used in fabricating the pipe. The metal reinforcing plate is galvanized sheet iron, perforated on 2-inch centers across the width of the plate and the rows of perforations are spaced so that a row is directly over each valley of the corrugations when the plate is placed in the pipe. The plate is fastened to the pipe by rivets placed on approximately 4-inch centers across each end and one rivet on each edge near the center of the plate. The ends of sheets are lapped approximately 2 inches at each section junction. With the metal reinforcement in place each length of pipe is coated with asphalt. The hot asphalt fills the valleys of the corrugations between the layer of felt and the pipe and an additional thickness of bituminous material is deposited on the surface of the metal plate, thus forming the metal reinforced asphalt pavement. The reinforcing plate in the pipe used on the Blue Ridge parkway had a spelter coating of 0.68 ounce per square foot and was approximately 26 gage.

In November 1939 a sample of bituminous-coated corrugated sheet metal pipe with reinforced asphalt pavement was submitted from section 2V–1 for examination of the paved invert and conformity with the specifications. On examining the invert it was observed that the crests of the corrugations, where the felt made

703454—46——1

TABLE 2.—*Inspection data for asphalt-coated galvanized metal culvert pipe with asphalt paved invert*

Section and station No.	Diameter of pipe	Length of pipe	Gage of metal	End Protection [1]		Grade of invert	Soil condition around culvert	Present flow of water through culvert	Condition of paved invert	Condition of coating inside pipe above invert			Condition of coating on outside of pipe	Condition of exposed metal
				Left	Right					Appearance	Loss	Adhesion		
Section 2M–1:	*Inches*	*Feet*				*Percent*								
18+68	24	126	14	HW	HW	55.8	Damp	None	Cracked, low loss	Checked	Low	Fair		Slightly rusted
44+00	18	99	16		HW		do	do	Badly cracked	Good		do	[2]	
55+00	18	50	16	DI		2.0	do	do	Cheeked	Good, some flow		do	[2]	Slightly rusted
63+73	18	44	16	DI		4.9	do	do	do	Good		do	[2]	
73+96	18	42	16	DI		.2	do	do	Cracked	Good, some flow		do	[2]	
84+85	24	149	14	HW		36.0	do	do	Good	Good		do	[2]	
98+84	18	176	14	HW		37.1	do	do	Cracked	do		do		
112+05	18	113	16	DI	HW	47.1	do	do	Good	Good, some flow		do		
124+67	18	39	16	DI	HW	5.7	do	do	Slightly cracked	Good		do	[2]	
141+94	18	50	16	DI		15.6	do	do	Badly cracked, loose	do		do	[2]	
152+23	18	60	16		DI	6.3	do	Slight	Badly cracked	do		do		
168+11	18	52	16	HW	DI	17.4	do	None	Good	do		do		
185+73	18	162	14	HW	HW	21.8	do	Slight	Checked	do		do		
204+50	24	83	14	HW	HW	7.1	do	None	Good	Good, some flow		do		
221+50	18	43	16	HW	DI	10.3	do	do	do		Low	do		
243+64	18	42	16		DI	3.3	do	do	Badly cracked, loose	Good		do	[2]	
255+00	18	53	16	DI		1.7	do	do	Cracked		Medium [3]		[2]	Good.
266+00	18	38	16	DI	HW	6.7	do	do	Slightly checked	Good		Fair		
293+78	18	112	16		DI	35.0	do	do	Badly cracked, some loose		Low	do	[2]	
306+47	18	58	16	DI		6.2	do	do	Badly cracked	Good		do	[2]	
328+35	18	78	16	HW	HW	26.9	do	do	Slightly cracked	Checked		do		
351+00	30	218	12	HW		48.9	do	Slight	do	Good, some flow		do	[2]	
356+57	30	182	12	HW		32.5	do	do	Slightly cracked, low loss	do		do	[2]	
375+57	18	144	14	HW		52.0	do	None	Good	Good		do	[2]	
385+28	24	128	14	DI	HW	48.9	do	do	do	do		do	[2]	
399+80	18	45	16	DI		15.1	do	do	Badly cracked, some loose	do		do	[2]	
Section 2U–2:														
287+00	18	65	16		HW	39.0	do	Slight	Cracked, loose, high loss [3]	do		Poor	[2]	Slightly rusted
306+50	24	32	14	HW	DI	12.2	do	do	Cracked, loose, loss [3]	do		do		
326+50	18	40	16		DI	.2	do	do	do	Checked	Low	do	[2]	
343+75	18	56	16		DI	36.6	do	None	Badly cracked, loose	do		do	[2]	
346+90	24	114	14		HW	38.0	do	do	Cracked, loss [3]	Good		do	[2]	
361+00	18	54	16		DI	17.0	do	do	Cracked, loss		Low	do	[2]	
380+00	18	41	16	HW	DI	9.1	do	do	Cracked		do	do		
396+00	18	34	16	HW	DI	4.4	do	Slight	Badly cracked, low loss	Good		do		
419+75	18	46	16	DI	HW	13.6	do	Small	Cracked	do		do		
450+00	18	91	16		HW	46.6	do	None	Cracked, high loss [3]		High [3]	do	[2]	Rusted.
537+95	18	49	16	DI	HW	20.4	do	do	Checked, low loss	Good		do		

[1] HW=Head wall. DI=Drop inlet.
[2] Where exposed, the outside coating was badly checked and brittle. Where covered by fill, coating was badly checked and had very poor adhesion to metal.
[3] Loss was throughout entire length.

TABLE 3.—*Inspection data for asphalt-coated asbestos-bonded metal culvert pipe with asphalt paved invert*

Section and station No.	Diameter of pipe	Length of pipe	Gage of metal	End protection [1]		Grade of invert	Soil condition around culvert	Present flow of water through culvert	Condition of paved invert	Condition of coating inside pipe above invert			Condition of coating on outside of pipe	Condition of exposed metal
				Left	Right					Appearance	Loss	Adhesion		
Section 2M-2:	*Inches*	*Feet*				*Percent*								
425+94	18	67	16		DI	32±	Damp	None	Cracked	Good		Good	(2)	
440+06	18	47	16		HW	1±	do	do	do	Cheeked		Fair	(2)	
464+00	18	60	16	DI		5±	do	do	do	Good		Good	(2)	
495+44	24	94	14	HW			do	do	do		Low		(2)	Asbestos intaet.
505+19	18	89	16	DI		47±	do	do	Transverse craeks entire length	Good		Good	(2)	
514+50	18	43	14	DI		5±	do	do	Sligbtly eracked	do		do	(2)	
526+65	18	57	16		DI	5±	do	do	Cracked, loose	Checked	Low		(2)	Do.
535+80	18	50	16		HW	8±	do	do	Craeked	Good		Good	(2)	
550+84	18	68	16		HW	9±	do	do	do	do		do	(2)	
578+25	24	44	14	HW		10±	do	do	Cracked entire length	Checked	Low		(2)	Do.
594+45	24	154	14	HW		35±	do	Medium	Cracked	do	do		(2)	
Section 2N-1:														
9+99	18	122	14	HW		19±	do	None	[3]		do		(2)	Do.
20+79	18	49	16	HW		6±	Dry	do	Cracked	Good		Good	(2)	
43+50	18	48	16	DI		3.0	Wet	Slight	Checked, some loose	do		do	(2)	
56+50	18	107	14		HW	34±	Damp	None	Checked	do		do	(2)	
69+36	18	78	16		DI	48±	do	do	do	do		do	(2)	
79+75	18	89	16		HW	38.0	do	do	Badly craeked	do		do	(2)	
90+25	18	52	16		HW	24.1	do	do	Cracked	do		do	(2)	
114+50	18	85	16		DI	44.1	do	do	do	do		do	(2)	
142+29	18	176	16		HW	53.0	Wet	Medium	Cracked at inlet end, some loose	Checked		Poor	(2)	
151+30	24	225	14		HW		Damp	None	Good	Good		Good	(2)	
166+74	18	45	16		DI	7.5	do	do	Craeked, loose	do		do	(2)	
176+45	18	63	16		DI	3.9	do	do	Cracked	do		do	(2)	
194+00	18	85	16		HW		do	do	Badly cracked	do		do	(2)	
207+35	24	81	14		DI	5.4	do	do	Craeked, loose	do		do	(2)	
219+50	18	108	16		HW		do	Slight	do	do		do	(2)	
298+80	18	64	16	DI			do	None	Checked	do		do	(2)	
309+09	18	153	16	DI		57.7–59.5	do	do	Good	do		do	(2)	
317+92	18	67	16	HW		4.1	do	Small	Checked	do		do	(2)	
331+00	18	96	16	HW			do	None	do	do		do	(2)	
341+79	30	140	12	HW			do	do	Checked entire length	do		do	(2)	
354+51	30	74	12	HW		26.4	do	Small	do	Good, some flow		do	(2)	
382+30	18	50	16	DI		17.0	do	None	Checked	Good		do	(2)	
391+24	18	56	16	DI		3.5	do	do	do	do		do	(2)	
401+50	24	46	14	DI		10.5	do	do	do	Good, some flow		do	(2)	
412+49	24	69	14	HW		29.1	do	do	do	Good		do	(2)	
440+75	18	80	16	DI		39.3	do	do	Low loss, spalling	do		do	(2)	Do.

[1] HW=Head wall. DI=Drop inlet.
[2] Where exposed, the outside coating was badly checked and brittle. Where covered by fill, coating was badly checked and had very poor adhesion to metal.
[3] Culvert filled with debris; invert could not be inspected.

TABLE 4.—*Inspection data for asphalt-coated asbestos-bonded metal culvert pipe with asphalt paved invert*

Section and station No.	Diameter of pipe	Length of pipe	Gage of metal	End Protection [1]		Grade of invert	Soil condition around culvert	Present flow of water through culvert	Condition of paved invert	Condition of coating inside pipe above invert			Condition of coating on outside of pipe	Condition of exposed metal
				Left	Right					Appearance	Loss	Adhesion		
Section 2P-2:	*Inches*	*Feet*				*Percent*								
406+39	18	62	16		DI	3.0	Damp	None	Good		Medium	Good	(2)	Asbestos intact.
408+50	18	98	16		DI		do	Small	Badly cracked	Checked	High		(2)	Do.
417+50	18	48	16		HW	3.0	do	None	Cracked	Good		Good	(2)	
428+99	18	55	16		DI	28.1	do	do	Good	do		do	(2)	
449+63	18	42	16		HW	3.0	do	do	do		Low		(2)	
466+50	24	52	14		HW	17.7	do	Slight	Badly cracked, loose [3]	Good		Good	(2)	
477+37	24	46	14		HW	12.3	do	Small	Cracked	do		do	(2)	
501+99	18	58	16		HW	20.4	do	None	do	do		do	(2)	
517+50	18	48	16		HW	2.9	do	do	Good	do		do	(2)	
530+54	18	48	16		HW	3.1	do	do	Checked	do		do	(2)	
544+00	18	78	16		DI	40.0	do	do	Good	do		do	(2)	
567+50	18	46	16		DI	23.7	do	Slight	Badly cracked, loose	do		do	(2)	
587+22	18	46	16		HW	4.2	do	None	Cracked, loose	do		Poor	(2)	
602+50	18	38	16		HW	4.1	do	do	Good [4]	Good [4]			(2)	Badly rusted in invert.
633+95	18	46	16		HW	21.4	do	Medium	Good	Good		Poor	(2)	
642+82	18	36	16		HW	6.1	do	None	Cracked	do		Good	(2)	
654+75	18	75	16		DI	32.6	do	do	Good	do		do	(2)	
664+27	24	72	14		HW	34.7	do	do	Good [4]	Good [4]			(2)	Good.
686+50	30	82	12		HW	30.7	do	do	Good	do	Low		(2)	
696+39	24	40	14		HW	3.4	do	do	Badly cracked	Good		Good	(2)	
710+00	18	76	16		DI	2.8	do	do	do	do		do	(2)	
719+66	30	118	12		HW	31.2	do	do	Good	Good, some flow		do	(2)	
728+90	18	56	16		DI	3.0	do	do	Checked	Good		do	(2)	
739+03	18	36	16		DI	5.2	do	do	Good	do		Poor	(2)	
759+97	30	84	12		HW	19.2	do	Slight	do	Good, some flow		Good	(2)	
767+90	18	78	14		DI	32.5	do	None	do	Good		do	(2)	
796+95	18	103	16		HW	10.0	do	do	Checked	do		do	(2)	
807+25	18	67	16		DI	3.0	do	do	Good	Good, some flow		do	(2)	
816+67	18	89	16		HW		do	do	Checked	Checked		do	(2)	
828+00	18	49	16		DI	28.9	do	do	Cracked	Good		do	(2)	
839+00	18	82	16		DI	43.8	do	do	do	do		do	(2)	

[1] HW = Head wall. DI = Drop inlet.
[2] Where exposed, the outside coating was badly checked and brittle. Where covered by fill, coating was badly checked and had very poor adhesion to metal.
[3] At left end one length of asphalt-coated galvanized metal pipe was used; remainder was asbestos-bonded pipe.
[4] At left end one length of uncoated galvanized pipe was used; remainder was asbestos-bonded pipe and appeared to be in good condition.

TABLE 5.—*Inspection data for asphalt-coated galvanized metal culvert pipe with reinforced asphalt invert*

Section and station No.	Diameter of pipe	Length of pipe	Gage of metal	End protection [1]		Grade of invert	Soil condition around culvert	Present flow of water through culvert	Condition of asphalt coating in invert			Condition of exposed metal reinforcing plate	Condition of coating inside pipe above invert			Condition of coating on outside of pipe
				Left	Right				Appearance	Loss	Adhesion		Appearance	Loss	Adhesion	
Section 2N–2:	*Inches*	*Feet*				*Percent*										
447+75	18	115	16	HW		37.7	Damp	None	Badly broken	Medium	Poor	Loose at ends, some rusting	Good	None	Fair	(2)
462+52	18	72	16	DI		11.1	Wet	Small	do	do	do	Some rusting	do	do	do	(2)
466+53	18	60	16	HW		27.9	Damp	do	Good	None	Fair		do	do	do	(2)
474+50	18	46	16	HW	DI	16.3	do	None	Badly broken	Medium	Poor	Intact, no rusting	Checked	do	do	
485+04	18	43	16	HW	DI	10.0	do	do	Checked	None	Fair		Good	do	do	
490+89	18	100	16		DI	45.0	do	Small	Badly broken	High	Poor	Loose at ends	do	do	do	(2)
504+28	24	79	14		HW	38.9	do	do	do	do	do	Intact, no rusting	do	do	do	(2)
514+63	24	50	14		DI	3.6	do	None	Good	None	do		do	do	do	(2)
533+88	18	61	16	HW	DI	35.6	Wet	do	Cracked	do			do	do	do	
535+75	24	77	14	HW	DI	24.8	do	Medium	Badly broken	Very high	Poor	Loose at ends, slight rusting	do	do	Poor	
537+71	30	70	12	HW	DI	20.4	Damp	Slight	Cracked	Medium	do	Loose at ends, pipe ¼ blocked	do	do	Fair	
544+32	18	57	16		DI	23.5	Wet	Small	Badly broken	do		Loose, badly rusted	do	do	do	(2)
576+28	30	48	12	HW	DI	7.9	Damp	do	Cracked	do		Intact, no rusting	do	do	do	
596+18	30	70	12	HW	HW	28.6	do	do	Good	None			do	do	do	(2)
617+64	30	80	12		DI	50.8	do	do	Badly broken	High	Poor	Loose, turned up at end	do	do	do	(2)
643+57	24	92	14		DI	36.6	do	do	Good	None			do	do	do	(2)
676+50	24	54	14	HW	DI	27.2	do	Large	do	do			do	do	do	
Section 2P–1:																
201+00	18	42	16		HW		Damp	None	Badly broken	Medium		Intact, no rusting	do	do	do	(2)
216+00	18	42	16		HW		do	do	Good	None			do	do	do	(2)
341+20	18	59	16				do	Medium	do	do	Poor		do	do	do	(2)
354+00	18	105					do	Small	do	do			do	do	do	(2)
381+00	30	134	12				do	Medium	Badly broken	High		Intact, no rusting	do	do	do	(2)
392+55	18	123			HW		do	None	(3)				do	do	do	(2)
399+45	18	48	16		HW		do	Slight	Badly broken	Medium		Intact, no rusting	do	do	do	(2)
Section 2V–1:																
140+17	24	104	14		HW	35.1	do	None		Low	Good	Intact, some rusting	do	do	Good	(2)
143+00	18	37	16		DI	4.0	do	do	Cracked	None	do		do	do	do	(2)
153+70	18	95	14		HW	21.7–33.0	do	do		Slight	do	Intact, no rusting	do	do	do	(2)
166+65	18	96	14		DI	66.6	do	do		High		do	do	do	do	(2)
174+35	24	73	14		HW	30.8	do	do		do		Loose, turned up at ends	do	do	do	(2)
191+38	18	49	16		DI	10.0	do	Slight	(3)				do	do	do	(2)
204+50	18	48	16		DI	5.0	do	None	Broken	Medium		Intact, badly rusted	do	do	do	(2)
223+46	24	36	14	DI		7.5	Wet	Medium		High			do	do	Poor	
245+25	18	71	16	HW		14.1–22.0	do	do		do			do	do	do	
262+70	18	93	14	HW		37.5	Damp	None	Checked	None	Poor		do	do	do	
274+38	18	99	14		HW	17.0–19.1	do	Medium		Medium	do		do	do	do	(2)

[1] HW=Head wall. DI=Drop inlet.
[2] Where exposed, the outside coating was badly checked and brittle. Where covered by fill, coating was badly checked and had very poor adhesion to metal.
[3] Outlet end of culvert partially filled and condition of paved invert could not be examined.

FIGURE 1.—TYPICAL CONDITION OF FLOW FROM CRESTS OF CORRUGATIONS IN THE TOP QUARTER INSIDE THE CULVERT PIPE.

contact, were not coated with asphalt. Also that the underside of the reinforcing plate, which was in contact with the felt, was not uniformly coated. Although the metal plate was perforated the space between the felt and the plate was too small to permit the hot asphalt to completely fill it.

LARGE NUMBER OF FIELD INSTALLATIONS INSPECTED

Because of the large number of culvert pipes it was not considered necessary to inspect all of them. To get a general idea of the condition of the pipes it was planned to inspect every third installation. This plan was not adhered to strictly because of inaccessibility of some of the outlets or because some of the culverts were filled with debris. The plan of inspection was altered in some instances to include culverts of special interest.

A separate report form was used for each culvert inspected to record data pertaining to the type and condition of the installation.

The date of installation, diameter and length of culvert pipe, gage of metal, type of end protection, and grade of the pipe invert were obtained from the records for each section of road. The soil surrounding the culvert was observed only as to its moisture content. The quantity of water flowing through the pipe was noted. Since inspection was made at a relatively dry time of the year the flow was considered continuous if there was water flowing through the pipe when inspected, and the flow was estimated on a comparative basis.

The condition of the paved invert and of the coating above the pavement and on the outside of the pipe was determined. Where there had been a loss of coating either in the paved invert or on the walls of the pipe, the condition of the exposed metal was observed. Drop inlets on many of the culverts made it difficult to make a thorough examination of the coating and pavement of the upper ends. Where possible, inspections were made from both ends or by crawling through the pipe.

DESCRIPTIVE TERM DEFINED

The data from the inspection report for each culvert were tabulated for each project and combined for each of the three general types of bituminous-coated pipe used. The inspection data are given in tables 2, 3, 4, and 5. Explanation of the terms used in these tables is given below.

Good.—Very little or no checking, no cracking, and no apparent loss of asphalt coating.

TABLE 6.—*Summary of conditions of paved inverts and coating on inside walls of all culverts inspected*

Section No.	Total culverts inspected	Number of culverts with paved invert				Number of culverts with inside walls		
		Good	Checked, no loss	Cracked or broken, no loss	Loss of coating	Good	Checked, no loss	Loss of coating
Asphalt-coated galvanized metal pipe with asphalt paved invert:								
2M–1	26	7	4	13	2	21	1	4
2U–2	11	0	0	3	8	6	1	4
Asphalt-coated asbestos-bonded pipe with asphalt paved invert:								
2M–2	11	0	0	11	0	6	1	4
2N–1	[1] 26	2	12	10	1	24	1	1
2P–2	31	15	4	12	0	26	1	4
Asphalt-coated galvanized metal pipe with reinforced asphalt invert:								
2N–2	17	5	1	1	10	16	1	0
2P–1	[1] 7	3	0	0	3	7	0	0
2V–1	[1] 11	0	1	1	8	11	0	0

[1] Outlet end of one culvert was filled; invert was not inspected.

Checked.—Fine hair cracks of insufficient width to permit large infiltration of water and silt. In the asphalt pavement checking usually occurred directly over the crests of the corrugations and often extended the full width of the invert. On the inside walls above the pavement and on the outside of the pipe, checking generally had the pattern of alligator skin.

Cracked.—This term is used to describe the condition of the asphalt pavement and denotes cracks of sufficient width to permit rapid infiltration of water and silt. In some culverts cracks often exceeded one-half inch in width for the full depth of the asphalt pavement. Cracking generally occurred above the crests of the corrugations, although in some pipes there were large longitudinal cracks. In some pipes where the pavement was severely cracked, the infiltration of water and silt had caused a loss of adhesion between the asphalt and the metal with subsequent loosening of pieces of the pavement.

Broken.—This term is used to describe the condition of the asphalt coating on the reinforcing plate (where used) and signifies severe cracking and breaking of the coating into small pieces.

Loss.—In the invert, loss was usually caused by pieces of the asphalt pavement first cracking and then being loosened and dislodged by flowing water.

On the inside walls above the pavement, loss appeared to be due to either erosion, impact, or stripping of the coating from the metal by the action of water.

Adhesion.—Adhesion of the asphalt to the metal was not determined accurately but a comparative indication was obtained by noting the ease with which the coating could be removed with a putty knife.

Flow.—In some culverts a displacement of the asphalt coating by flow was noted. This condition occurred at the top of the pipe and was in the form of strings of asphalt hanging from the crests of the corrugations. Flow is illustrated in figure 1, which shows the coating cut from the corrugations, reassembled, and photographed.

In the two types of pipes where the pavement consisted wholly of asphalt, the most deterioration of the

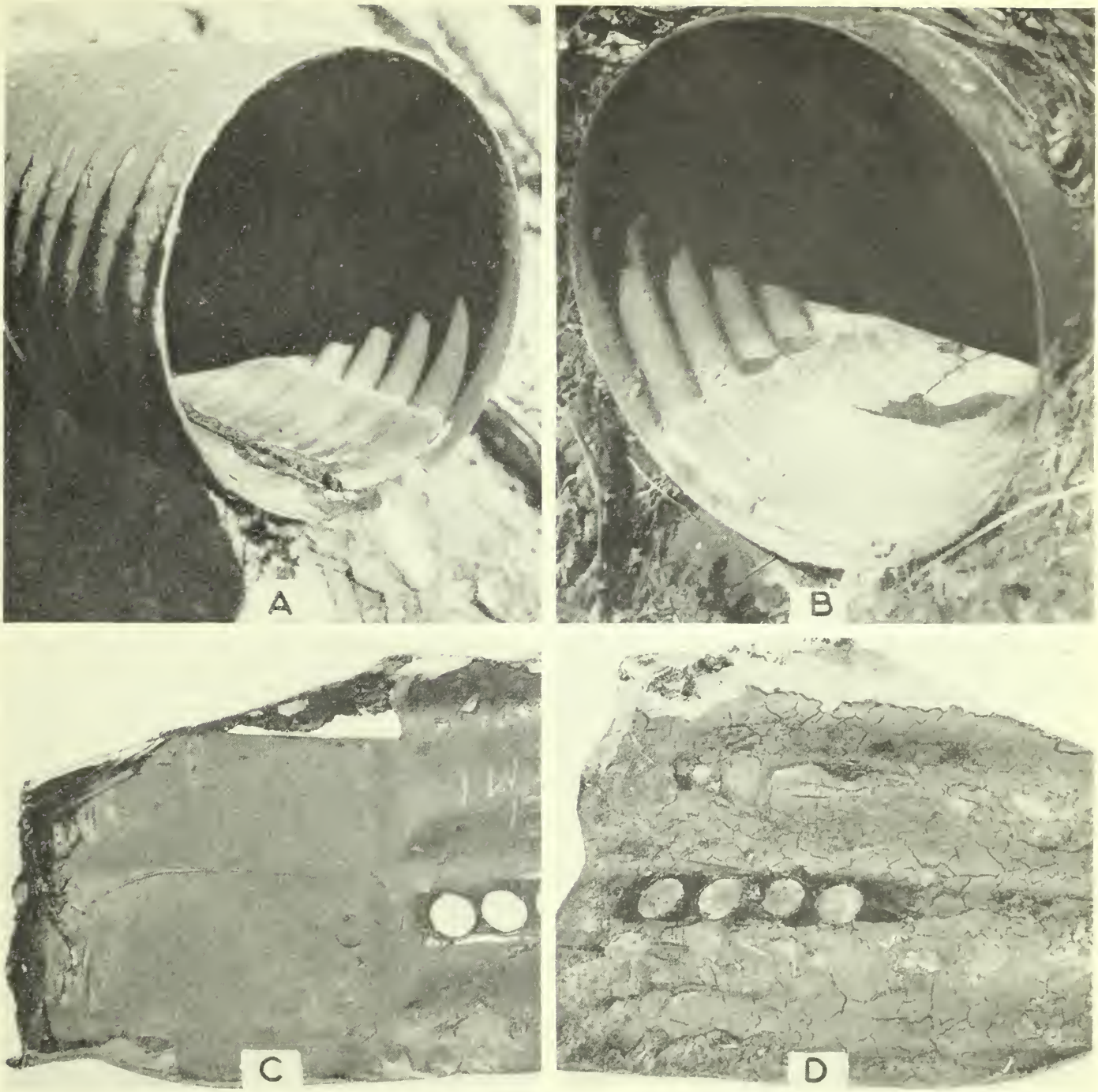

FIGURE 2.—BITUMINOUS-COATED GALVANIZED METAL PIPE: (A) SEVERE LONGITUDINAL CRACKING; (B) CRACKING OF ASPHALT PAVEMENT; (C) INSIDE VIEW OF SAMPLE CUT FROM OUTLET END OF PIPE. CONDITION APPEARS TO BE GOOD BUT ADHESION TEST (ROUND SPOTS) SHOWS LOW ADHESION; AND (D) OUTSIDE OF SAMPLE SHOWN IN (C). THIS PORTION OF THE PIPE WAS IN CONTACT WITH SOIL. TESTS SHOWED POOR ADHESION.

pavement and coating on the inside walls was found to be at the outlet end of the culverts which was more exposed to sunlight and changes in temperature. The condition of the invert, inside and outside coating, and exposed metal given in tables 2, 3, and 4 are for the outlet end except where otherwise indicated. In the culvert pipe with the reinforced asphalt pavement the deterioration of the asphalt coating and metal in the invert was, in general, more uniform throughout the entire length of the pipe. Therefore, these conditions as shown in table 5 apply to the full length of the culvert pipe.

Table 6 gives a summary of the condition of the paved inverts and inside walls by road sections. There follows a discussion of the data on the performance of each type of pipe on the different sections of road.

PLAIN ASPHALT-COATED PIPE

Section 2M–1.—At the time of the inspection the asphalt coating and pavement were considered to be giving adequate protection. No culvert pavements or coatings had failed but conditions were found that may be indicative of future failures. Twenty-six culverts

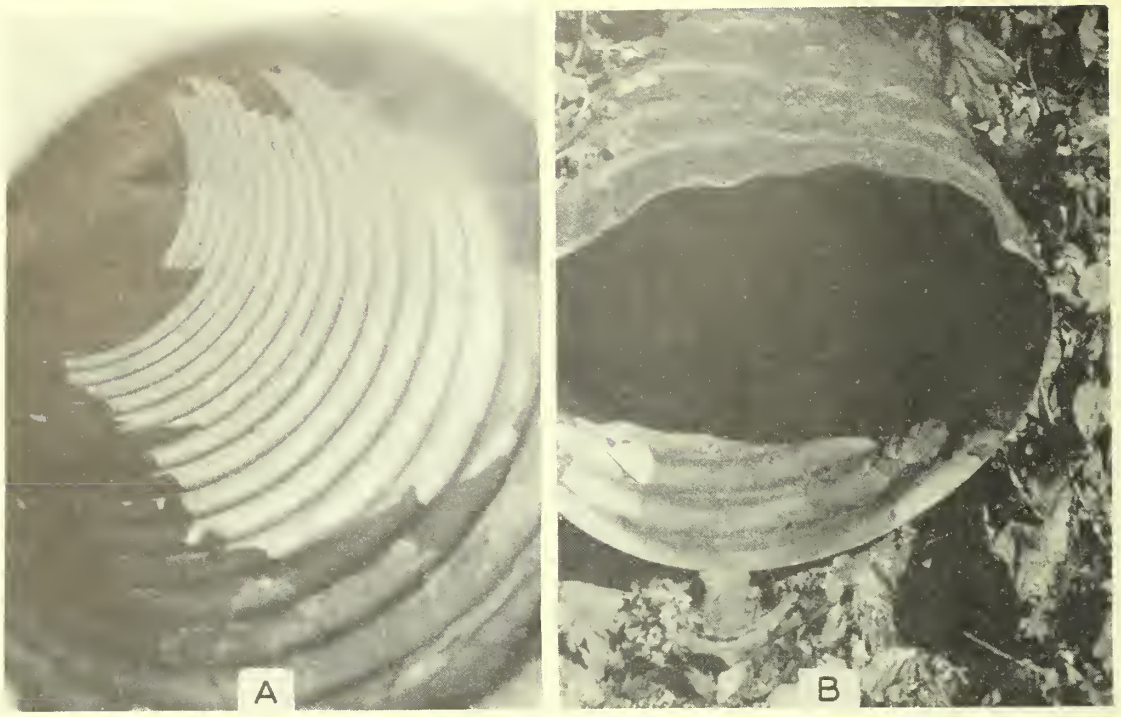

FIGURE 3.—A, ASPHALT COATING STRIPPED FROM WALL BY WATER SURGING THROUGH THE CULVERT ON 47 PERCENT GRADE; B, LOSS OF PAVEMENT FROM SAME CULVERT WITH RUSTING OF EXPOSED METAL.

were inspected. The pavements in 7 culverts were rated good, 4 were checked, 13 were cracked only, and 2 showed sufficient loss of coating to expose the metal. Typical conditions are shown in figure 2.

The coatings on the inside of the pipes above the inverts were generally in good condition but appeared to have only fair adhesion to the metal. In this respect, 21 culverts were rated good, 1 was checked only, and 4 showed some loss of coating.

Section 2U–2.—In general the condition of the culvert pipe installed on section 2U–2 was much worse than on section 2M–1 and for some culverts the coating was not considered to be giving adequate protection to the metal. The adhesion of the asphalt to the metal appeared to be poor. Of the 11 culverts inspected none of the paved inverts was rated as good, 3 were cracked, and 8 had lost some of the coating.

The inside coatings above the inverts also showed more deterioration than on section 2M–1. For 6 of the 11 culverts inspected the coatings were good, 1 was checked only, and 4 showed some loss of coating. One culvert, which had an extremely high loss of coating on the inside walls, is of special interest. This culvert was installed on a 25° skew on a grade of 47 percent with a head wall at the inlet end. It appeared that during heavy rainfall water ran down the gutter and into the pipe at high velocity. It struck the side of the pipe above the pavement at an angle and had stripped a large area of the asphalt coating from the side wall. This condition was repeated on alternate right and left sides of the pipe for a considerable distance down the pipe. Figure 3, A shows the high loss of coating from the inside wall near the inlet. The loss of asphalt coating from the pavement is illustrated in figure 3, B.

BITUMINOUS-COATED ASBESTOS-BONDED PIPE

Bituminous-coated asbestos-bonded metal culvert pipe was used on sections 2M–2, 2N–1, and 2P–2. Although there was considerable checking and cracking of the asphalt pavements near the outlet, all the inverts were giving good service. In comparison with sections 2M–1 and 2U–2 there were fewer cracked areas that were loose or easily removed. Where there was a loss of asphalt the asbestos sheet was generally intact and appeared to give some additional protection to the metal. The coatings on the inside walls above the pavements seemed to have better adhesion to the asbestos-bonded metal than did the asphalt on the plain galvanized metal on section 2M–1. A summary of the condition of the pavements and inside coatings in the culverts where the asbestos-bonded metal was used follows.

Section 2M–2.—Eleven culverts were inspected and all the inverts were found to be cracked but there was no loss of coating. The coating on the inside walls of 6 culverts was rated good, 1 was checked only, and on 4 there was some loss.

Section 2N–1.—Twenty-six culverts were inspected. The paved inverts of 2 were regarded as good, 12 were checked, 10 were cracked, 1 showed some loss of asphalt, and the condition of 1 invert could not be ascertained. The coating on the inside walls of 24 culverts was considered good, 1 showed some checking and 1 some loss of asphalt.

Section 2P–2.—Thirty-one culverts were inspected. The paved inverts in 15 were good, 4 were checked, and 12 were cracked. There was no loss of pavement in any of these pipes. In 26 of the culverts the coating on the inside walls was considered good, 1 was checked only, and 4 had some loss.

Typical conditions of the culvert pipe installed on sections 2M–2 and 2N–1 are shown in figures 4 and 5.

PIPE WITH REINFORCED PAVEMENT

Bituminous-coated corrugated sheet metal culverts with metal reinforced asphalt pavement were installed on sections 2N–2, 2P–1, and 2V–1. Before the inspec-

FIGURE 4.—BITUMINOUS-COATED ASBESTOS-BONDED PIPE SHOWING SEVERE CRACKING OF PAVEMENT AT OUTLET END.

tion some difficulty had been experienced by the maintenance men in keeping some of the culverts on section 2N–2 in serviceable condition due to failure of the inverts. Inspection of the culverts revealed that in some culverts there had been a high loss of coating from the paved invert, exposing the metal reinforcing plate to the action of water and erosion. Subsequent action of water and erosion through the pipes caused the exposed reinforcing plates to loosen and pull away from the rivets holding them to the pipe. Because of the light weight of the reinforcement the loosened ends were turned up and eventually formed blocks within the pipes. This condition usually occurred at the upper end of the section lengths where they were connected by bands.

A summary of the conditions found in culverts where the metal reinforced pavement was used follows.

Section 2N–2.— Of the 17 culverts examined there were 5 in which the pavement was considered good, 2 were checked or cracked only, and in 10 culverts the asphalt was cracked or broken and showed considerable loss. In 6 culverts the exposed metal reinforcing plates were partially turned up, and in 4 culverts they were rusted. The coatings on the insides of the pipes above the pavement were in good condition and appeared to have fair adhesion to the metal.

Section 2P–1.—Since construction of this section had not been completed only 7 culverts were inspected. The paved inverts of 3 were in good condition, 3 were badly broken with some loss of coating, and 1 invert could not be examined. The metal reinforcement in all culverts was in place. The inside walls were in good condition and the coatings appeared to have fair adhesion to the pipe.

Section 2V–1.—Eleven culverts were inspected. Eight had lost coating from the inverts and the exposed plates in 2 of these were rusted. The asphalt coating

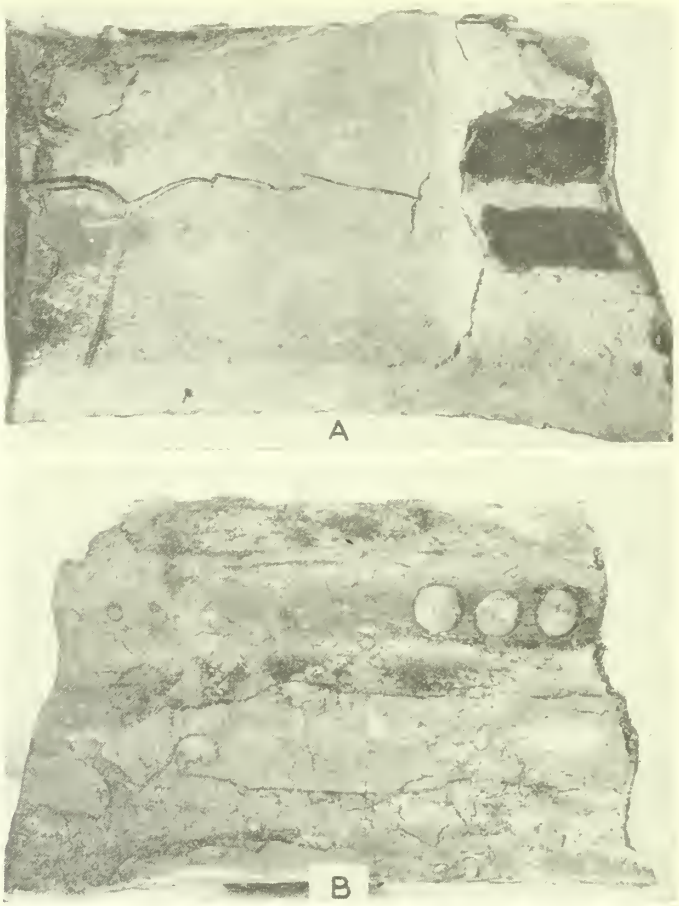

FIGURE 5.—SAMPLE TAKEN FROM OUTLET END OF BITUMINOUS-COATED ASBESTOS-BONDED METAL PIPE. (A) INSIDE SHOWS CRACKED PAVEMENT; (B) OUTSIDE SHOWS GENERAL CONDITION OF COATING WHICH HAS BEEN IN CONTACT WITH SOIL SURROUNDING CULVERT.

in the inverts of 2 culverts was checked or cracked. The metal reinforcing plates appeared to be in place. The condition of the invert of one culvert could not be ascertained. The inside walls of all culverts inspected were in good condition but in some culverts the coating appeared to have better adhesion to the metal than in others.

The conditions of some of the culvert pipe installed on sections 2N–2, 2P–1, and 2V–1 are illustrated by figure 6.

OUTSIDE COATINGS SEVERELY CHECKED

The asphalt coating on the outside of the culvert pipes was found to be in the same general condition on all the sections inspected. Where the outlet ends of the culverts were exposed, the asphalt coating on the outside was hard and severely checked but on most culverts there was good adhesion to the metal. On some culverts there was loss of coating, which appeared to be due to impact of rocks from the slopes above the outlets.

Where it was possible to dig away the cover material and examine the asphalt coating under the fill, it was found to be severely checked and to have very low or no adhesion to the metal. At the time of the inspection the soil surrounding the culverts usually was damp or wet and it is probable that it remains in this condition most of the time.

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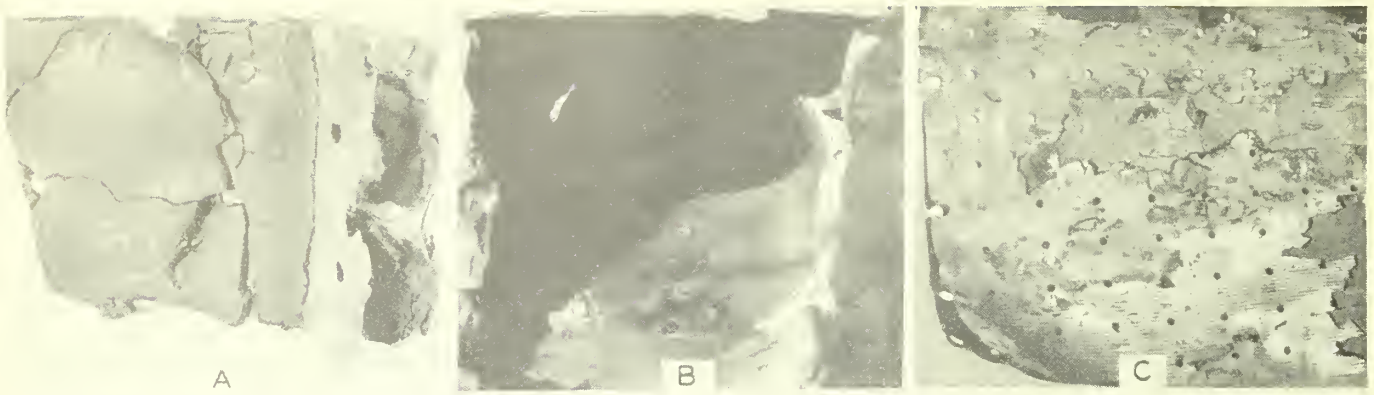

FIGURE 6.—METAL-REINFORCED ASPHALT PAVING. (A) SAMPLE FROM OUTLET END SHOWING LOOSENING OF REINFORCEMENT; (B) COATING ON REINFORCING PLATE BADLY BROKEN WITH SOME LOSS; AND (C) SECTION OF REINFORCING PLATE SHOWING LOSS OF COATING AND RUSTING OF EXPOSED METAL.

LABORATORY TESTS MADE ON FIELD SAMPLES

Samples, representing the three general types of bituminous-coated metal pipe, were cut from the culverts for analysis in the laboratory. Samples of asphalt were also taken from the paved inverts or from the inside walls.

During the construction of Blue Ridge parkway, samples of culvert pipe had been submitted to the laboratory to be tested for conformity with the specifications. Since the only test of asphalt required by these specifications was determination of solubility in carbon disulphide, other characteristics of the asphalt at the time of installing the pipe are not known. Because of this lack of information, a satisfactory comparison between the characteristics of the original asphalt coating and the samples taken at this time cannot be made.

A complete analysis of all asphalt samples was made and the results are shown in table 7. Considering the effects of variations in exposure of the different samples, the average results of tests on samples from the same fabricator of the coated pipe, as shown in table 8, indicate that different asphalts were used by the three fabricators. Analyses of seven typical blown asphalts of the type used or intended for use as pipe coating materials were also made and the test values are given in table 9. These asphalts were submitted either by the fabricator of the bituminous-coated pipe or by the producer of the asphalt. Although the analyses of the field samples given in tables 7 and 8 do not correspond exactly to those of any of the seven typical asphalts, both groups of asphalts are of the same general type.

In 1945 the American Railway Engineering Association [1] issued a specification for bituminous-coated corrugated metal pipe and arches. In this specification a test was proposed for the determination of the adhesion of bituminous coating to metal, the determination to be made on samples taken from the coated pipe, delivered or about to be delivered to the purchaser. To make

[1] Committee Report, American Railway Engineering Association, Bul. 451, February 1945, vol. 46, No. 451, p. 493.

TABLE 7.—*Analyses of samples of asphalt coating taken from culvert pipes*

Section and station No.	Penetration, 100 gm, 5 sec.			Ductility 5 cm. per min. 25° C.	Softening point	Specific gravity, 25°/25° C.	Solubility in CS_2			Bitumen insoluble in 86° B. naphtha	Oliensis test	
	At 15° C.	At 25° C.	At 35° C.				Bitumen	Organic insoluble	Inorganic insoluble		Character of spot	Naphtha-xylene equivalent
				Cm.	*° C.*		*Percent*	*Percent*	*Percent*	*Percent*		*Percent*
Section 2M-1:												
18+68	29	44	64	2.0	98.5	0.999	99.71	0.10	0.19	32.15	Positive	5-10
98+84	31	47	71	2.0	95.5	.999	99.48	.12	.40	30.82	do	10-15
141+94	27	42	64	1.75	96.5	.999	99.67	.10	.23	31.20	do	5-10
152+23	27	41	62	1.75	100.0	1.000	99.73	.21	.06	32.51	do	5-10
204+50	23	35	55	2.0	99.0	1.007	99.08	.19	.73	34.17	do	10-15
Section 2U-2, 306+50	23	36	51	1.75	99.0	.988	99.80	.16	.04	31.82	Negative	
Section 2M-2, 526+65	26	41	61	2.0	97.3	1.003	99.41	.08	.51	32.77	Positive	5-10
Section 2N-1:												
79+75	27	40	60	1.75	98.3	.999	99.82	.13	.05	32.29	do	0-5
166+74	30	45	68	1.75	98.5	1.000	99.72	.12	.16	30.74	do	15-20
207+35	26	40	60	1.75	101.5	1.001	99.35	.16	.49	31.84	do	5-10
Section 2P-2:												
719+66	28	43	63	2.0	99.5	.998	99.68	.15	.17	32.82	do	5-10
466+50	25	40	63	1.75	98.5	1.000	99.66	.11	.23	34.83	do	5-10
587+22	26	39	59	1.75	100.8	.998	99.75	.10	.15	33.36	do	0-5
Section 2N-2:												
447+75	18	26	38	[1].5	114.0	1.020	99.12	.37	51	34.73	do	45-50
474+50	20	34	48	[1] 1.0	101.0	1.011	99.64	.17	.19	33.94	do	45-50
504+28	17	27	42	2.25	100.0	1.010	99.55	.21	.24	35.07	do	5-10
535+75	21	29	40	[1].5	119.5	1.009	99.30	.19	.51	35.15	do	50-55
544+32	20	31	49	2.25	98.3	1.012	99.73	.11	.16	34.11	do	15-20
617+64	22	33	49	2.5	96.0	1.012	99.72	.15	.13	34.86	do	10-15
Section 2V-1:												
140+17	19	29	44	2.25	101.8	1.012	99.31	.20	.49	34.81	do	15-20
166+65	24	35	51	[1].5	109.3	1.013	99.66	.16	.18	32.91	do	45-50
245+25	29	43	62	1.75	101.3	1.009	99.77	.12	.11	31.34	do	40-45
262+70	23	33	47	[1].5	109.0	1.011	99.30	.25	.45	33.28	do	45-50

[1] Test specimen broke at smallest cross section.

TABLE 8.—*Average analysis of asphalt coatings used by the fabricators of the coated pipe*

Fabricator	Sections where used	Penetration, 100 gm. 5 sec. at—			Ductility at 25° C.	Softening point	Specific gravity, 25°/25° C.	Bitumen insoluble in 86° B. naphtha	Xylene equivalent
		15° C.	25° C.	35° C.					
					Cm.	° *C.*		*Percent*	*Percent*
A	2M–1, 2M–2, 2N–1, 2P–2	27	41	63	1.9	98.7	1.000	32.46	[1] 8.3
B	2U–2	23	36	51	1.8	99.0	.988	31.82	0
C	2N–2, 2V–1	21	32	47	1.4	105.0	1.012	34.02	[1] 34.0

[1] Average value of the mean naphtha-xylene equivalent.

TABLE 9.—*Analyses of typical blown asphalts from various sources*

Identification No.	Penetration 100 gm., 5 sec. at—			Ductility 5 cm. per min. 25° C.	Softening point	Specific gravity, 25°/25° C.	Solubility in CS_2			Bitumen insoluble in 86° B. naphtha	Oliensis test	
	15° C.	25° C.	35° C.				Bitumen	Organic insoluble	Inorganic insoluble		Character of spot	Naphtha-xylene equivalent
				Cm.	° *C.*		*Percent*	*Percent*	*Percent*	*Percent*		*Percent*
1	21	31	45	2.0	110.0	1.011	99.86	0.14	0	37.16	Positive	12–16
2	32	49	72	2.0	107.2	.981	99.64	.20	.16	33.24	do	0–4
3	29	45	69	2.3	100.0	1.007	99.86	.14	0	32.32	do	24–28
4	21	35	59	4.0	83.3	1.014	99.74	.26	0	32.62	Negative	
5	19	30	46	2.5	95.0	1.003	99.84	.16	0	33.34	do	
6	23	38	62	3.3	90.6	1.013	99.83	.17	0	33.11	Positive	8–12
7	29	42	61	2.0	97.8	.983	99.89	.11	0	30.34	Negative	

this test, a round brass disk one-half square inch in area is embedded in the asphalt coating and pulled off by applying a load at a uniform rate. The ability of the coating to retain its adherence to the pipe is indicated by the amount of coating remaining on the metal after pulling off the disk. This amount of coating retained by the metal is expressed as a percentage of the total area under the disk. For 100-percent retained coating the asphalt breaks in cohesion and denotes good adhesion, zero percent denotes poor adhesion and values between 100 and zero percent coating retained are proportional between good and poor adhesion.

For some time the Public Roads Administration has been studying the adhesion of bituminous coatings to metal and a test similar to that proposed by the American Railway Engineering Association has been used. In this study not only the amount of coating removed by the adhesion disk is noted but also the load required to pull the disk off is determined. It has been found that the adhesion load may be as significant as the amount of coating retained in evaluating the adhesion of coating materials.

The samples of bituminous-coated metal culverts from the Blue Ridge parkway were tested for adhesion of the asphalt coating to the metal. The results are given in table 10, and include adhesion values for the coating on the outside of the pipe, the inside walls above the pavement, and the coating in the invert. With the exception of the asbestos-bonded metal the adhesion values, as measured by the percentage of coating retained, indicate the asphalt to have little adhesion to the metal for all the samples and for the different areas tested on the same sample. In the tests on asbestos-bonded metal, failures occurred in the asbestos sheet and the coating can be considered as having good adhesion. The adhesion values as measured by the load required to pull the disk off indicate the variable adhesiveness of the asphalt coating not only on different pipes but also on different areas of the same pipe.

As indicated by the values of the adhesion load, the adhesion of the asphalts to the metal was, in general, best in the paved inverts and poorest on the outside walls of the pipe. The adhesion load values for the sample from section 2N–1 are nearly the same for the outside, inside, and the invert, but the latter two values were determined on asbestos-bonded metal and should not be compared with those made on plain galvanized metal. The adhesion values for the sample from section 2N–2 show the asphalt coating to have lower adhesion to the reinforcing plate than to either the outside of the pipe or to the inside of the pipe directly under the reinforcement. The adhesion of the coating to the reinforcing plate from section 2N–2 also was very low. These low values may account for the high loss of coating from the inverts in those culverts having the reinforcing plates.

The asphalt coating from the samples was heated and used to coat various kinds of metal plates. Adhesion tests were made on these coated plates to get some indication of the initial adhesion of the various asphalts to the kind of metal on which they were used in the pipe. The results of these tests are given in table 10.

The adhesion load values for the metal plates coated with the reheated asphalts are in most cases higher than the highest value obtained on the corresponding samples taken from the culverts. The amount of coating retained by the metal also is higher on the recoated galvanized and reinforcing plates than on the corresponding pipe samples. Assuming that the adhesion values for the recoated plates are indicative of the adhesion of the original coating to the pipe, there has been a loss of adhesion while the culverts were in service.

There is no correlation between any of the laboratory tests made on the asphalt coatings from the culvert pipes and the adhesion of these coatings. In the study of the adhesion of pipe coating materials now in progress it has been found that certain characteristics of asphalts may be indicative of their adhesion and should be considered in selecting asphalt to insure high initial adhesion and the retention of that adhesion to the metal after exposure in service.

TABLE 10.—*Results of adhesion tests on pipe samples and on laboratory-coated metal plates using asphalts taken from the pipe samples*

Section number	Station number	Coated pipe samples from culverts				Laboratory-coated plates using asphalts from pipe samples		
		Area tested for adhesion	Type of coated metal	Adhesion load	Coating retained	Type of surface coated	Adhesion load	Coating retained
				Pounds	*Percent*		*Pounds*	*Percent*
2M-1	98+84	Outside	Galvanized	11.4	None	Galvanized	31.4	100
		Inside wall	do	13.8	None			
		Paved invert	do	28.9	None			
2M-1	152+23	Outside	Galvanized	14.9	None	Galvanized	39.2	100
		Paved invert	do	39.5	50			
2M-2	526+65	Outside	Galvanized	9.9	None	Galvanized	42.7	100
		Inside wall	Asbestos-bonded	12.8	(1)	Asbestos-bonded	33.8	(1)
		Paved invert	do	24.3	(1)			
2N-1	207+35	Outside	Galvanized	18.0	None	Galvanized	36.1	100
		Inside wall	Asbestos-bonded	18.9	(1)	Asbestos-bonded	39.8	(1)
		Paved invert	do	18.0	(1)			
2N-2	544+32	Outside	Galvanized	26.8	None	Galvanized	51.6	100
		Paved invert [2]	Reinforcing plate	15.8	10	Reinforcing plate	32.0	20
		Paved invert [3]	Galvanized	38.0	35			
2N-2	535+75	Paved invert [2]	Reinforcing plate	12.0	20			

[1] Asbestos sheet was split by test.
[2] Coating on surface of reinforcing plate.
[3] Coating on corrugated metal under reinforcing plate.

SUMMARY

The data obtained in this study seem to warrant the following summary:

1. In general, the most severe deterioration of the asphalt coating and pavement was found at the outlet ends of the pipes.

2. The amount of flow of water did not seem to bear any consistent relation to the degree of deterioration occurring in the coating and pavement.

3. In general, the slope of the pipe had little influence upon the deterioration of the asphalt coating and pavement.

4. The asphalt coating on the outside of the culvert pipe was usually checked and had very little adhesion to the metal.

5. The metal-reinforced asphalt-paved inverts showed the greatest amount of deterioration under service conditions.

6. The culvert pipes in which asbestos-bonded metal was used appeared to have the greatest resistance to deterioration. Even though there was some loss of asphalt coating the asbestos sheet appeared to give added protection to the metal.

7. The better condition of the asphalt coating on section 2M–1 as compared with that on section 2U–2 seems to indicate that the durability of the coating depends to a considerable degree upon the type of asphalt used.

Because of the limited information available on the durability of bituminous-coated sheet metal culvert pipe and because of the short time the culverts inspected had been in service, it seems advisable to continue periodic inspections of these and other installations.

A STUDY OF COARSE-GRADED BASE-COURSE MATERIALS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by Edward A. Willis, Senior Highway Engineer

THIS REPORT[1] is the seventh in a series describing investigations of granular materials for surface and base-course construction. Previous reports described laboratory and circular track tests on sand-clay and sand-clay-gravel materials, nonplastic granular materials with admixtures of water-retentive chemicals, the chemical treatment of chert-gravels, lignin binder in the treatment of crusher-run materials, and volcanic cinders.

In the report on the tests of sand-clay-gravel materials,[2] it was indicated that the plasticity index of the binder soil is less critical in coarse-graded mixtures having less than 40 percent passing the No. 10 sieve than in similar mixtures containing larger amounts of soil mortar. The purpose of this investigation was to study the effect of variations in plasticity index, ranging from nonplastic to well above the generally accepted A. A. S. H. O. specification limit of 6, on the stability of base-course mixtures having essentially the same grading and all with less than 40 percent passing the No. 10 sieve.

Five mixtures were prepared by combining crushed limestone, sand, silica dust (ground quartz), and clay. The mixtures were designed to have a grading meeting A. A. S. H. O. specification for a type B–1 base-course material. The plasticity index was varied by varying the amount of silica dust and clay in the fraction of the mixture passing the No. 200 sieve. One mixture was nonplastic while the other four mixtures were designed to have plasticity indexes of approximately 5, 10, 15, and 20. The gradings, liquid limits, and plasticity indexes of the five mixtures and the A. A. S. H. O. specification limits for type B–1 base-course materials are shown in table 1.

TABLE 1.—*Grading and physical constants of base-course materials*

Grading	Type B–1 base course, A. A. S. H. O. specification	As constructed					At end of test				
		Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5	Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5
Passing, sieve:	*Percent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*
1-inch	100	100	100	100	100	100	100	100	100	100	100
¾-inch	70–100	90	88	89	90	93	94	89	91	89	93
⅜-inch	50–80	56	59	60	58	61	63	60	63	60	59
No. 4	35–65	39	42	42	44	43	48	45	46	40	41
No. 10	25–50	32	33	33	38	34	36	34	33	28	30
No. 40	15–30	18	19	18	21	19	19	20	19	16	18
No. 200	5–15	10	10	10	11	10	10	10	10	9	9
Dust ratio [1]	Less than 66⅔	56	53	56	52	53	53	50	53	56	50
Liquid limit	Not over 25	(2)	19	23	29	34	(2)	18	25	31	32
Plasticity index	Not over 6	(2)	4	9	17	21	(2)	3	12	18	18

[1] Dust ratio $=100\times\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}$

[2] Nonplastic.

TRAFFIC TESTS MADE ON OUTDOOR CIRCULAR TRACK

The mixtures were subjected to traffic tests on the circular track used in previous studies of water-retentive chemicals as admixtures with nonplastic road-building materials.[3] Wheel loads were applied by 30-by 5-inch high-pressure tires requiring an inflation pressure of 80 pounds per square inch. The load imposed by each wheel was 800 pounds until near the end of the test when it was increased to 1,000 pounds.

Distributed traffic was used for compacting the mixtures and was obtained by gradually shifting the rotating beam longitudinally with respect to its axis of rotation causing the wheels to pursue alternately expanding and contracting spiral courses covering the entire track area.

Concentrated traffic was applied in the actual tests by locking the sliding pivot of the beam in such position that the wheels pursued two concentric circular courses whose center lines were about 2½ inches on either side of the center line of the track.

Since the mixtures used in this investigation contained a large percentage of coarse angular particles, the standard A. A. S. H. O. compaction test could not be used to determine the optimum moisture content for compaction. The materials for each section, after thorough dry mixing, were wetted with just enough water to make them workable and again thoroughly mixed. Representative samples were taken from the mixture as prepared for each section. The moisture contents were determined and vibratory compaction tests were made. Data from these tests as well as on moisture content and density obtained at the end of the traffic tests are shown in table 2.

After the materials for each section had been prepared by wetting and mixing, they were placed in the track in two approximately equal layers to obtain a total base thickness of 6 inches. The bottom lift was hand-tamped in place and further compacted by 400 wheel-trips of distributed traffic at slow speed, obtained by rotating the beam at 9.6 revolutions per minute. The

TABLE 2.—*Moisture-density relations*

Section No.	Vibrator compaction test				Moisture content when placed	Data obtained at end of test				
	Dry density	Volume aggregate	Volume voids	Moisture content if voids are filled		Dry density	Moisture content	Volumetric composition		
								Solids	Water	Air
	Pounds per cubic foot	*Percent*	*Percent*	*Percent by weight*	*Percent by weight*	*Pounds per cubic foot*	*Percent by weight*	*Percent*	*Percent*	*Percent*
1	146	85.9	14.1	6.0	5.3	148.2	3.3	86.7	7.8	5.5
2	141	82.6	17.4	7.7	5.2	146.3	4.0	85.5	9.4	5.1
3	141	82.5	17.5	7.7	5.0	146.5	4.7	85.6	11.0	3.4
4	136	79.7	20.3	9.3	5.8	144.9	5.1	84.7	11.8	3.5
5	139	81.3	18.7	8.4	4.8	142.6	6.3	83.4	14.4	2.2

[1] This report is based on data collected, under the supervision of the author, by Mr. Walter T. Hamilton, junior highway engineer, prior to his entrance into the armed services.

[2] A Study of Sand-Clay-Gravel Materials for Base-Course Construction, by C. A. Carpenter and E. A. Willis, PUBLIC ROADS, vol. 20, No. 1, March 1939.

[3] Studies of Water-Retentive Chemicals as Admixtures with Nonplastic Road-Building Materials, by E. A. Willis and C. A. Carpenter, PUBLIC ROADS, vol. 20, No. 9, November 1939.

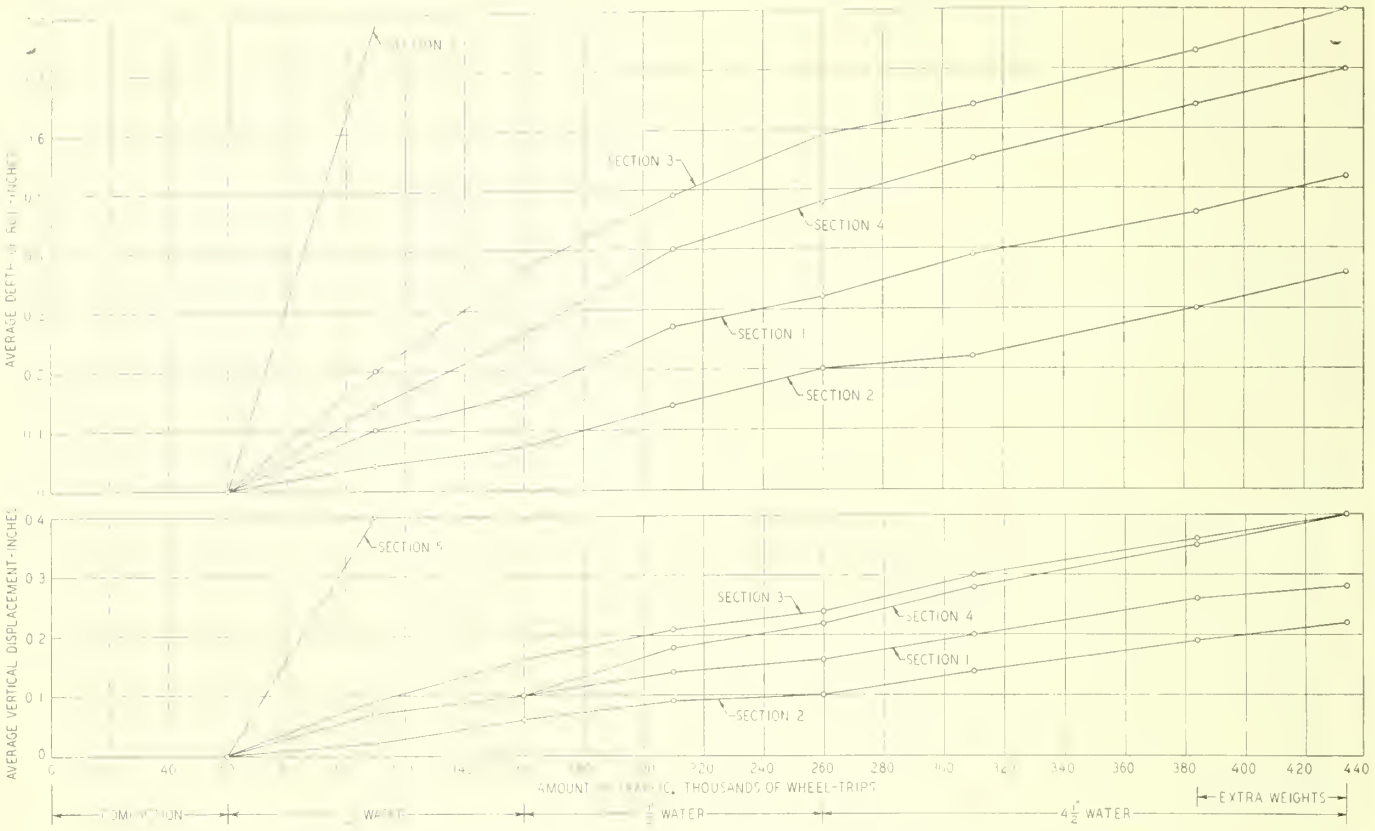

FIGURE 1.—SURFACE DISPLACEMENT OF SECTIONS OF THE TRACK DURING THE COURSE OF THE TEST.

top lift then was placed and tamped, and distributed traffic continued. After 270 wheel-trips, the speed was increased to medium or 14.0 revolutions per minute.

The compaction operations were started during damp weather. Under this condition the track did not become smooth and firm. Two soft spots developed in section 4, which showed pronounced movement under traffic, and operations were suspended temporarily. With the advent of good drying weather traffic was resumed. Compaction proceeded rapidly and the base was soon ready for surface treatment. The total amount of distributed traffic required to compact the base was 40,800 wheel-trips.

After compaction, the base was allowed to dry for several days. Then a prime coat consisting of approximately 0.3 gallon per square yard of light tar was applied and allowed to cure. Finally, a surface treatment consisting of approximately 0.4 gallon of hot-application bituminous material and a cover of 50 pounds per square yard of aggregate of ¾-inch maximum size was applied. Distributed traffic was used to compact the surface treatment. After 400 wheel-trips at slow speed, the speed was increased to medium and continued until the entire surface was smooth, well sealed, and showed no movement.

Compaction of the surface treatment required 19,200 wheel-trips bringing the total distributed traffic to 60,000 wheel-trips.

TESTING WITH CONCENTRATED TRAFFIC

After construction and compaction had been completed, initial profile measurements were made, water

TABLE 3.—*Schedule of operations and behavior of test sections*

Operation	Water level above top of subbase	Traffic	Behavior				
			Sec. 1 (1)	Sec. 2 (P. I.—4)	Sec. 3 (P. I.—9)	Sec. 4 (P. I.—17)	Sec. 5 (P. I.—21)
Distributed traffic at slow speed, compacting bottom lift	*Inches*	*Wheel-trips* 0–400	Stable	Stable	Stable	Stable	Stable.
Distributed traffic at slow speed, compacting top lift		400–670	do	do	do	Unstable	Do.
Distributed traffic at medium speed, compacting top lift		670–40,800	do	do	do	Stable	Do.
Distributed traffic at slow speed, compacting surface treatment		40,800–41,200	Good	Good	Good	Good	Good.
Distributed traffic at medium speed, compacting surface treatment		41,200–60,000	do	do	do	do	Do.
Testing with concentrated traffic	½	60,000–110,000	do	do	do	Unstable	Failed.
Do	1½	110,000–160,000	do	do	do	Good	
Do	2½	160,000–210,000	do	do	Fair	Unstable	
Do	2½	210,000–260,000	do	do	do	Fair	
Do	4½	260,000–310,000	Fair	do	Failed	Failed	
Do	4½	310,000–384,000	Unstable	do			
Testing with concentrated traffic and extra weights	4½	384,000–434,000	Failed	Slightly unstable			

[1] Plasticity index, nonplastic

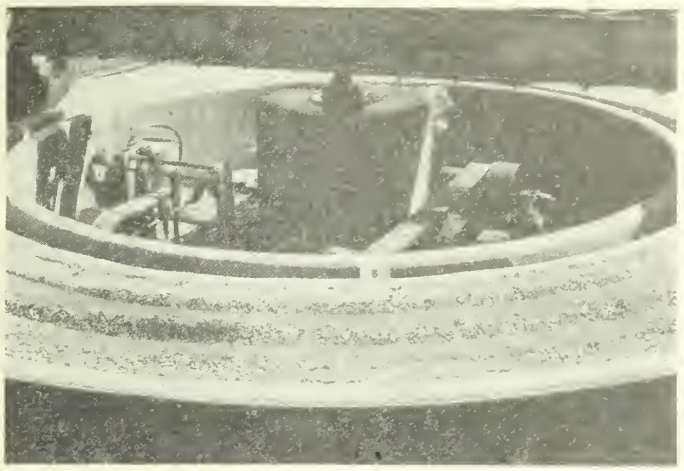

FIGURE 2.—APPEARANCE OF SECTION 5 AFTER 110,000 WHEEL-TRIPS.

was introduced into the subbase and allowed to rise to a height of one-half inch above the top of the subbase, and testing with concentrated traffic was started. As shown in the schedule of operations, table 3, the water elevation was maintained at one-half inch above the top of the subbase during the first 100,000 wheel-trips of concentrated traffic. The water was then raised to 2½ inches for a similar period and finally was raised to 4½ inches above the top of the subbase. This high water elevation was maintained throughout the remainder of the test period. After 384,000 wheel-trips, the load on each wheel was increased from 800 to 1,000 pounds. Testing was discontinued after a total of 434,000 wheel-trips had been applied.

The behavior of the materials was judged by the appearance of the sections at various stages of the tests supplemented by measurements of vertical displacement of the surface. Previous reports have described the transverse [4] and longitudinal [5] profilometers with which the measurements were made.

Transverse profiles were plotted and the area between the initial and each succeeding profile was measured with a planimeter. The measured area divided by the track width of 18 inches gave the vertical displacement. The average displacement for the two stations on a section gave the average vertical displacement for that section.

The area between the initial and each succeeding longitudinal profile made for each wheel lane was measured for each section. That area divided by the length of the wheel lane gave the depth of rutting, and the average for the two-wheel lanes in a section gave the average depth of rutting for that section.

Surface displacements of the sections at various stages of the test are shown in figure 1. The schedule of traffic applications and changes in water elevations with notations on the behavior of the five test sections are given in table 3.

In reports on previous investigations on the same circular track, it has been stated that an average vertical displacement of 0.25 inch is sufficient to cause marked damage to the bituminous wearing course. It was also found that, with the ground water elevation one-half inch above the top of the subbase, concentrated traffic provided a condition sufficiently severe to enable the identification of the definitely unsatisfactory base-course materials.

Section 5 clearly failed to give satisfactory performance as measured by these standards. This section, which had a plasticity index of 21, began to show movement under traffic soon after water was admitted to the subbase. At 110,000 wheel-trips, the average vertical displacement had reached 0.4 inch and the section was considered as "failed" (see table 3). Figure 2 shows the condition of the section at this time.

It was necessary to repair section 5 before the test on the other sections could be continued. The surface was scarified and crushed limestone and tar added. The section was then hand-tamped until firm. Profilometer measurements on the section were discontinued.

BASES WITH HIGH PLASTICITY INDEXES INFERIOR

The nonplastic base course in section 1 remained in good condition until exceptionally severe conditions were imposed near the end of the test. When the water was raised to 4½ inches above the top of the subbase after 260,000 wheel-trips, this section began to show slight signs of distress although the average vertical displacement (see fig. 1) did not reach 0.25 inch until about 370,000 wheel-trips. Damp spots began to appear on the surface and slight movement under traffic was noted. Increasing the wheel load to 1,000 pounds had no appreciable effect as far as could be observed and the average displacement continued at a uniform rate.

At 434,000 wheel-trips, when traffic testing was discontinued, section 1 was rated as failed primarily because the average vertical displacement was in excess of 0.25 inch. Figure 3 shows the condition of sections 1, 2, and 3 at the end of the test.

Section 2, which had a plasticity index of 4, behaved slightly better than section 1 at all stages of the test. Introduction of water, even at the maximum elevation of 4½ inches above the top of the subbase, had little effect on the stability. The only noticeable change was a gradual consolidation in the wheel tracks. No upward movement of material was observed between the wheel tracks or at the edges of the track. As shown in figure 1, the displacements measured on section 2 were less than those measured on any other section. At the end of the test, the average depth of rut was approximately 0.36 inch and the average vertical displacement was approximately 0.22 inch. After 434,000 wheel-trips, the conclusion of traffic, the section was still in serviceable condition.

Section 3, with a plasticity index of 9, was stable throughout the compaction period (60,000 wheel-trips). During the testing with concentrated traffic with water one-half inch above the top of the subbase, the section remained in good condition although there was a gradual development of ruts as shown by the displacement measurements plotted in figure 1.

After 160,000 wheel-trips, the water level was raised to 2½ inches and concentrated traffic continued. During this phase of the testing, displacement and rutting continued at a uniform rate.

At about 200,000 wheel-trips slight movement of the surface and base could be observed with the passage of each wheel. This movement continued throughout the remainder of the test although it was never of sufficient magnitude to disrupt the surface.

[4] Circular Track Tests on Low-Cost Bituminous Mixtures, by C. A. Carpenter and J. F. Goode, PUBLIC ROADS, vol. 17, No. 4, June 1936.
[5] See footnote 2, page 239.

FIGURE 3.—CONDITION OF SECTIONS 1, 2, AND 3 AT THE END OF THE TESTS.

The average vertical displacement of section 3 was approximately 0.24 inch at 260,000 wheel-trips, the conclusion of testing with water at the 2½-inch elevation. This was more than for any other section except section 5 which had failed at about 100,000 wheel-trips. Displacements continued uniformly under concentrated traffic with the water level raised to 4½ inches above the top of the subbase. By the time traffic had reached 300,000 wheel-trips, the average vertical displacement was well over 0.25 inch, mud and water began appearing through cracks in the surface, and the section was rated failed although traffic was continued over the section to the end of the test without difficulty.

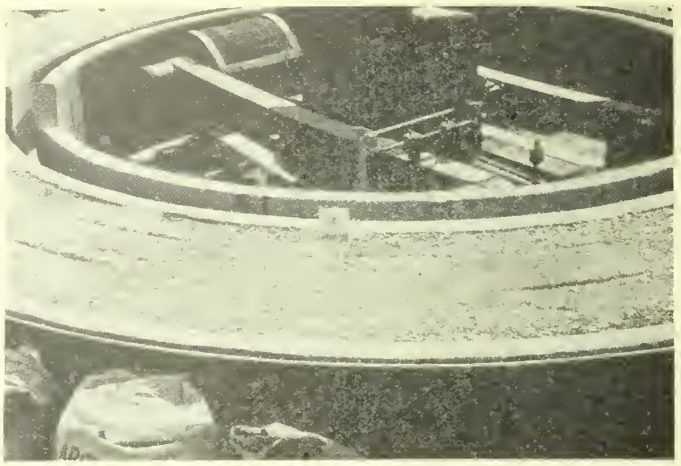

FIGURE 4.—SECTION 4 AFTER 110,000 WHEEL-TRIPS. NOTE SIGNS OF LOCAL FAILURE WHILE THE SECTION AS A WHOLE SHOWS VERY LITTLE DISPLACEMENT.

The behavior of section 4, which had a plasticity index of 17, was erratic. As previously noted, two soft spots developed in this section early in the compaction operation. Although these spots became firm before the surface treatment was constructed, they persisted as areas of local weakness throughout the early phases of testing with concentrated traffic. Immediately after the introduction of water into the subbase the passage of the wheels caused these spots to move visibly and to become depressed to such an extent that it appeared the section would fail.

As traffic continued, these areas became more stable. By the time 110,000 wheel-trips had been completed, the movement of the base was barely perceptible. At this time, the surface generally was in good condition with local rutting at the two weak points as shown in figure 4. No further change was observed until the water level was raised to 2½ inches above the top of the subbase at 160,000 wheel-trips.

The average vertical displacement in section 4 at 160,000 wheel-trips was less than that in section 3 at the same time. However, when testing was resumed with the water level raised to 2½ inches, the rate of average vertical displacement in section 4 was greater than that in section 3, and by the end of the test the average vertical displacements of both sections were equal and well in excess of 0.25 inch.

The two weak spots originally observed in section 4 again became unstable under traffic when the water was raised to 2½ inches above the top of the subbase. The movement in these areas gradually decreased. At 200,000 wheel-trips slight movement of the base was observed in the entire section. This general movement throughout section 4 continued until 260,000 wheel-trips, at which point the water level was raised to 4½ inches above the top of the subbase.

Section 4 failed as soon as traffic was resumed after increasing the water elevation. A mud-boil appeared at one of the soft spots at this time and the surface of the entire section moved under traffic. At 310,000 wheel-trips, the average vertical displacement of section 4 was almost 0.3 inch and the entire section was rated failed although traffic was continued to 434,000 wheel-trips in order to evaluate other sections. The appearance of section 4 at the end of traffic testing is shown in figure 5.

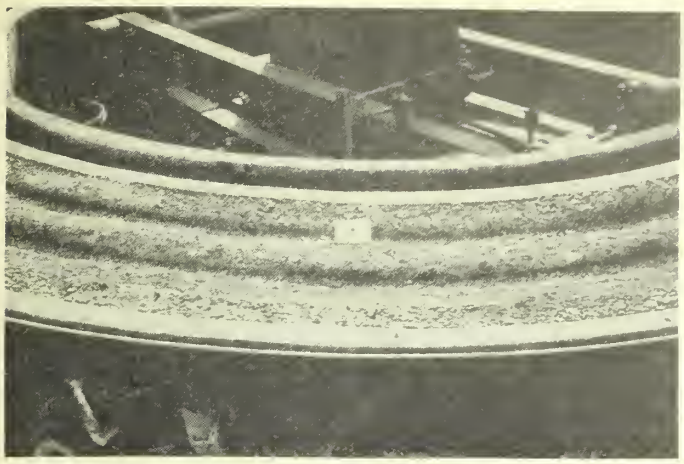

FIGURE 5.—SECTION 4 AT THE END OF THE TEST.

PLASTICITY INDEX IS A GUIDE TO PERFORMANCE

All five of the mixtures tested met the requirements for grading of the A. A. S. H. O. specification for type B–1 base-course material, but, as shown in table 1, only those placed in sections 1 and 2 met the plasticity index requirement.

The criteria used in evaluating these mixtures in the circular track tests were that an average vertical displacement of 0.25 inch was sufficient to cause marked damage to the bituminous wearing course and that concentrated traffic with water one-half inch above the top of the subbase provided a condition sufficiently severe to enable the identification of definitely unsatisfactory base-course materials.

Based on these criteria, the sections can be rated in order of superiority, as follows:

Section 2—Plasticity index 4	Excellent
Section 1—Nonplastic	Satisfactory
Section 3—Plasticity index 9	Border line
Section 4—Plasticity index 17	Unsatisfactory
Section 5—Plasticity index 21	Unsatisfactory

The behavior of section 2 was excellent throughout the tests. After 434,000 wheel-trips, including 174,000 wheel-trips with water 4½ inches above the top of the subbase, the average vertical displacement was still less than 0.25 inch. The dry density at the conclusion of the test (table 2) was 146.3 pounds per cubic foot and the moisture content was 4 percent. Section 1 behaved almost as well as section 2. The nonplastic material compacted under traffic somewhat more than did the mixture with a plasticity index of 4 used in section 2. As a result, the dry density at the end of the test was 148.2 pounds per cubic foot.

It was not until near the end of the test when extremely severe conditions were imposed that the average vertical displacement measured on section 1 exceeded 0.25 inch.

Base-course materials having gradings and plasticity indexes similar to those used in sections 1 and 2 would, therefore, be expected to give excellent service under all moisture conditions with those having low but definite plasticity indexes being slightly superior under the most adverse conditions.

Section 3 (plasticity index 9) was readily compacted and, under concentrated traffic, with water one-half inch above the top of the subbase, the average vertical displacement of this section was less than 0.2 inch. Therefore, the material cannot be rated as definitely unsatisfactory. However, displacement of the surface continued under continued concentrated traffic and exceeded 0.25 inch shortly after 260,000 wheel-trips when the water elevation was raised to 4½ inches above the top of the subbase. Although the final dry density of section 3 was 146.5 pounds per cubic foot, slightly higher than that of section 2, evidence of lateral movement was observed and mud and water were being forced through cracks in the surface near the end of the test. The section was therefore rated as border line. Materials having similar gradings and plasticity indexes would be expected to perform satisfactorily in well-drained areas, but would probably undergo sufficient displacement under adverse moisture conditions to cause failure of the surface.

Section 4 (plasticity index 17) gave trouble during compaction as well as periodically during the testing with concentrated traffic. Although the average vertical displacements measured on this section were less than those measured on section 3, except for the readings at the end of the test, the pronounced tendency of the material to become unstable each time the water elevation was raised is indicative of unsatisfactory performance under any but dry conditions.

Rutting started in section 5 (plasticity index 21) as soon as testing with concentrated traffic commenced with water one-half inch above the top of the subbase and progressed rapidly. The average vertical displacement was 0.4 inch at 110,000 wheel-trips at which time it was necessary to repair the section. Mixtures meeting A. A. S. H. O. grading requirements, but having plasticity indexes as high as this one, are therefore considered definitely unsatisfactory for use as base-course materials.

CONCLUSIONS

Based on the results of tests described in this report, it is concluded that:

1. Mixtures meeting the grading requirements of the A. A. S. H. O. specification for type B–1 base-course materials should also conform to the plasticity index requirement for best results under traffic even though the mixtures contain less than 40 percent of material passing the No. 10 sieve.

2. A material meeting the grading requirements and having a definite and measurable plasticity index within the specification limits is to be preferred to absolutely nonplastic materials having similar gradings and is decidedly superior to those having appreciably higher plasticity indexes.

3. Although the material having a plasticity index of 9 gave fairly satisfactory service except in the latter portion of the tests run with high water elevation, the possibility of the occurrence of conditions approaching complete saturation at some time during the life of a base course makes the present A. A. S. H. O. specification limit of 6 for the plasticity index a desirable if not a vitally necessary requirement.

TESTS OF CONCRETE CONTAINING WAYLITE

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by W. F. Kellermann, Senior Materials Engineer

A VARIETY OF MATERIALS are used as lightweight aggregates for concrete and are sold under different trade names. Probably the best known and most widely used material is known commercially as Haydite, an artificial aggregate made by burning shale or clay. Other lightweight aggregates are made by different processes. One of these is made by processing slag with steam in a centrifuge, the resulting product being known as Waylite.

When the tests reported herein were initiated, Waylite was being marketed both as a fine powder (practically 100 percent passing a No. 200 sieve), and as fine and coarse aggregate. It was decided to study both products, the first by using finely ground Waylite dust as a replacement for a portion of the portland cement in the mix, and the second by comparing some of the properties of concrete made with fine and coarse Waylite aggregate with those of comparable concrete made with natural materials.

Three series of tests were run. Series A was planned to determine the effect of substituting various percentages of Waylite dust for portland cement on the crushing and flexural strengths of concrete. In series B the effect of using Waylite fine and coarse aggregate on the unit weight and on the crushing and flexural strengths of concrete was determined by comparison with results obtained with natural aggregates. Series C was to determine the effect of using Waylite dust to replace portland cement on the resistance of concrete to attack by sulphates.

Natural river sand and crushed limestone were used as aggregates in series A and C and as aggregates for comparison with Waylite aggregate in series B.

All of the strength specimens in series A were cured in moist air and tested at 28 days. In series B, continuously moist-cured specimens were tested at 28, 180, and 360 days and, in addition, one group of specimens was cured the first 180 days in moist air and the remaining 180 days in laboratory air. This made it possible to compare the strengths of specimens cured the entire period in moist air with the strengths of specimens subjected to both moist air and laboratory air curing. The specimens used in series C were cut from beam specimens remaining after tests for flexural strength in series A.

For series A and C, four base mixes (mixes without Waylite dust) were designed with different water-cement ratios and a constant slump (2.5 to 3 inches). Water-cement ratios were selected as follows: 0.70, 0.80, 0.90, and 1.00, by volume. The corresponding cement factors ranged from 6.3 to 4.4 sacks of cement per cubic yard. For each base mix, three additional mixes were made in which Waylite dust was used to replace cement in the following percentages by weight: 25, 32, and 40. The substitution of Waylite dust for cement in the amounts used did not appear to affect the slump or ease of placement of the concrete in any way.

In series B data were obtained on the basis of both a comparable cement content and a comparable water-cement ratio. For the first condition two mixes for each aggregate combination were designed, one containing six sacks and the other seven sacks of cement per cubic yard. For the second condition, an additional mix was designed with natural aggregates having the same water-cement ratio as the 6-sack Waylite concrete.

TABLE 1.—*Chemical analysis of cement and Waylite dust*

Cement	Percent	Waylite dust	Percent
Silica	20.25	Silica	37.60
Alumina	6.70	Titanium oxide	.72
Ferric oxide	3.05	Alumina	11.82
Lime	62.70	Ferrous oxide	.32
Magnesia	3.68	Manganous oxide	1.72
Sulphuric anhydride	1.77	Calcium oxide	42.85
Loss on ignition	1.43	Calcium sulphide	3.17
		Magnesia	1.81
Tricalcium silicate	47	Insoluble residue	.38
Tricalcium aluminate	13		

TABLE 2.—*Grading and physical properties of aggregate and Waylite dust*

Sieve analysis	Natural sand	Crushed limestone	Waylite		
			Fine aggregate	Coarse aggregate	Dust
Percentage retained on:					
1½-inch sieve					
¾-inch sieve		60		4	
⅜-inch sieve		90		53	
No. 4 sieve	5	100		89	
No. 8 sieve	25	100		99	
No. 16 sieve	38	100	31	100	
No. 30 sieve	55	100	65	100	
No. 50 sieve	85	100	87	100	
No. 100 sieve	96	100	93	100	
No. 200 sieve					2.9
Fineness modulus	3.04	7.50	2.76	6.45	
Physical properties:					
Bulk specific gravity	2.59	2.75	1.90	1.10	[1] 2.86
Absorption (percent)	.8	.32	8.3	9.8	

[1] Apparent specific gravity.

Chemical analyses of the cement and Waylite dust are shown in table 1. Physical tests of the aggregates are given in table 2 while the mix data are shown in tables 3 and 4. The effect on the weight of the concrete of using Waylite fine and coarse aggregate is shown in the last column of table 4.

The concrete was mixed in a laboratory mixer with a capacity of 1¾ cubic feet and of the pan and paddle type, the pan being removable for cleaning. Test specimens were standard 6-inch by 12-inch cylinders and 6- by 6- by 21-inch beams. The absorption of the Waylite aggregate was very high, which is, of course, typical of aggregates of this type. For this reason all aggregates were immersed in water for 48 hours prior to mixing the concrete. However, the Waylite dust used in series A and C was not immersed as it was used as a substitute for cement.

All beam specimens were tested on an 18-inch span by the third point loading method, A. S. T. M. designation C 78.

The specimens used in series C to study the effect of Waylite dust upon the resistance of the concrete to alkali were 6-inch cubes sawed from the 6- by 6-inch beam specimens remaining after the strength tests of series A. Two cuts were made on each half beam so

TABLE 3.—*Mix data and unit weight of fresh concrete for series A and C*

Mix No.	Waylite replacement by weight [1]	Proportions by dry weight				Cement factor, base mix	Water-cement ratio, base mix	Total water content	W_v [4]	Weight of fresh concrete
		Cement	Waylite	Aggregate						
				Fine [2]	Coarse [3]					
	Pct.	*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*	*Sacks per cu. yd.*		*Gals. per cu. yd.*		*Lbs. per cu. ft.*
1(a)	none	94	none	188	331	6.3	0.70	32.7	0.162	152.8
1(b)	25	70.5	23.5	188	331			32.7	.162	152.8
1(c)	32	63.9	30.1	188	331			32.7	.162	152.8
1(d)	40	56.4	37.6	188	331			32.7	.162	152.8
2(a)	none	94	none	221	389	5.4	.80	32.5	.161	152.4
2(b)	25	70.5	23.5	221	389			32.3	.160	151.2
2(c)	32	63.9	30.1	221	389			32.5	.161	152.0
2(d)	40	56.4	37.6	221	389			32.7	.162	153.2
3(a)	none	94	none	250	442	4.9	.90	32.9	.163	153.2
3(b)	25	70.5	23.5	250	442			32.5	.161	151.6
3(c)	32	63.9	30.1	250	442			32.7	.162	152.0
3(d)	40	56.4	37.6	250	442			32.7	.162	152.0
4(a)	none	94	none	278	490	4.4	1.00	33.1	.164	152.0
4(b)	25	70.5	23.5	278	490			32.9	.163	151.6
4(c)	32	63.9	30.1	278	490			32.9	.163	151.2
4(d)	40	56.4	37.6	278	490			32.9	.163	151.2

[1] Waylite dust used to replace a portion of the cement in the percentages shown.
[2] Natural river sand with a fineness modulus of 3.04.
[3] Crushed limestone, maximum size 1½ inches.
[4] Volume of water per unit volume of concrete.

TABLE 4.—*Mix data and unit weight of fresh concrete for series B*

Mix No.	Proportions by dry weight			Type of aggregate		Actual cement factor	Water-cement ratio	Slump	Weight of fresh concrete
	Cement	Aggregates		Fine	Coarse				
		Fine	Coarse						
	Lbs.	*Lbs.*	*Lbs.*			*Sacks per cu. yd.*		*Ins.*	*Lbs. per cu. ft.*
1	94	198	349	Natural sand.	Limestone	6.0	0.74	2.1	152
2	94	155	300	do	do	7.0	.67	3.2	154
3 [1]	94	278	490	do	do	4.4	1.00	2.1	152
4	94	122	122	Waylite	Waylite	6.0	1.04	1.6	93
5	94	104	104	do	do	7.1	.85	1.2	97

[1] Mix designed to have approximately the same water-cement ratio as mix No. 4, containing Waylite aggregates.

as to expose aggregate on two of the six surfaces. When 42 days old the cube specimens were completely immersed in a 10-percent alkali solution of 5 percent magnesium sulphate and 5 percent sodium sulphate. During the 14-day interval between 28 and 42 days the specimens were kept continuously wet. Periodic weighings were made after immersion in alkali solution from which the losses in weight were determined.

DISCUSSION OF SERIES A

Results of strength tests of series A are given in table 5 and in figures 1 and 2. The flexural strengths shown in table 5 and plotted in figure 1 show how the percentage of Waylite dust in the mix affected the strength of concretes of varying cement content. Water-cement ratio as used in the discussion of series A refers only to the water-cement ratio of the base mix of each group, mix containing no Waylite dust. For total water content of mixes containing Waylite dust see table 3. It will be observed from figure 1 that in concrete for which the base water-cement ratio was 0.70, mixes No. 1(a) to 1(d), inclusive, the flexural strength

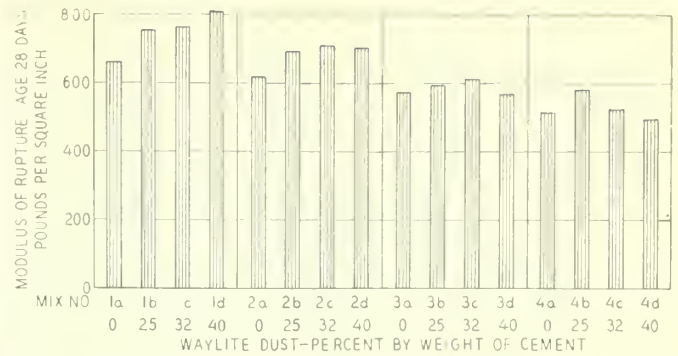
MODULUS OF RUPTURE AGE 28 DAYS
POUNDS PER SQUARE INCH
800
600
400
200
0
MIX NO
1a 1b c 1d 2a 2b 2c 2d 3a 3b 3c 3d 4a 4b 4c 4d
0 25 32 40 0 25 32 40 0 25 32 40 0 25 32 40
WAYLITE DUST–PERCENT BY WEIGHT OF CEMENT

FIGURE 1.—EFFECT OF USING WAYLITE DUST AS A REPLACEMENT FOR PORTLAND CEMENT ON THE FLEXURAL STRENGTH OF CONCRETE, SERIES A.

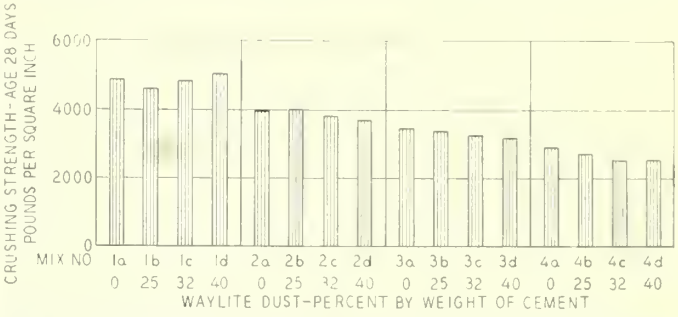
CRUSHING STRENGTH-AGE 28 DAYS
POUNDS PER SQUARE INCH
6000
4000
2000
0
MIX NO
1a 1b 1c 1d 2a 2b 2c 2d 3a 3b 3c 3d 4a 4b 4c 4d
0 25 32 40 0 25 32 40 0 25 32 40 0 25 32 40
WAYLITE DUST–PERCENT BY WEIGHT OF CEMENT

FIGURE 2.—EFFECT OF USING WAYLITE DUST AS A REPLACEMENT FOR PORTLAND CEMENT ON THE CRUSHING STRENGTH OF CONCRETE, SERIES A.

increased as the percentage of Waylite dust increased, the maximum increase being 23 percent with 40-percent replacement. In the case of the group for which the water-cement ratio of the base mix was 0.80, mixes No. 2(a) to 2(d), inclusive, the maximum increase in strength was 15 percent with 32-percent replacement. In the case of the next group, mixes 3(a) to 3(d), inclusive, the maximum increase in strength of 7 percent was also found with the 32-percent replacement, whereas in the case of the last group, mixes 4(a) to 4(d), inclusive, the 25-percent replacement gave the highest strength, a 13-percent increase over the straight portland cement mix. In this instance the 40-percent replacement was slightly lower in strength than the straight portland cement mix, the 32-percent replacement showing about

TABLE 5.—*Results of strength tests of concrete at 28 days for series A* [1]

Mix No.	Waylite dust replacement by weight	Water-cement ratio of base mix	Modulus of rupture	Modulus of rupture ratio [2]	Crushing strength	Crushing strength ratio [2]
	Percent		*Pounds per square inch*	*Percent*	*Pounds per square inch*	*Percent*
1 (a)	None	0.70	661	100	4870	100
1 (b)	25		755	114	4590	94
1 (c)	32		763	115	4830	99
1 (d)	40		810	123	5070	104
2 (a)	None	.80	620	100	3980	100
2 (b)	25		696	112	4020	101
2 (c)	32		715	115	3820	96
2 (d)	40		705	114	3710	93
3 (a)	None	.90	575	100	3470	100
3 (b)	25		596	104	3400	98
3 (c)	32		614	107	3270	94
3 (d)	40		572	99	3200	92
4 (a)	None	1.00	518	100	2910	100
4 (b)	25		585	113	2740	94
4 (c)	32		526	102	2510	86
4 (d)	40		500	97	2530	87

[1] Specimens continuously moist cured until tested. Each value is the average of three tests.
[2] Strength for base mix taken as 100 percent.

TABLE 6.—*Results of strength tests of concrete for series B* [1]

Mix No.	Actual cement factor	Type of aggregate	Modulus of rupture at age indicated in days				Crushing strength at age indicated in days			
			28	180	360	360 [2]	28	180	360	360 [2]
	Sacks per cu. yd.		*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
1	6.0	Natural sand and limestone.	692	766	744	682	4,400	5,140	5,340	5,910
2	7.0	do	688	794	761	736	5,100	5,820	5,640	6,160
3	4.4	do	560	616	566	618	3,220	3,880	3,440	3,780
4	6.0	Waylite	351	374	347	136	2,140	2,310	2,820	2,390
5	7.1	do	438	432	410	154	2,720	2,840	2,980	2,920

[1] Specimens continuously moist cured until tested unless otherwise noted. Each value is the average of two tests.
[2] Moist curing 180 days followed by 180 days curing in air. Specimens tested dry.
[3] Mix designed to have approximately the same water-cement ratio as mix No. 4, containing Waylite aggregates.

the same strength. It may be stated that the replacement of part of the portland cement by Waylite dust had a beneficial effect upon the flexural strength of the concrete at 28 days, this effect being more marked for concretes with low water-cement ratios (high cement factors).

It will be observed from the data that the amount of Waylite dust giving the maximum beneficial effect varied with the water-cement ratio, the lower the water-cement ratio (richer the mix), the greater the percentage of Waylite dust permissible. The flexural strength of the concrete was not adversely affected by a replacement of cement by Waylite dust up to 32 percent by weight. For concrete in which the base water-cement ratio is 0.90 or less, this percentage may be increased to 40 percent. In studying these results it should be noted that the tests were made at 28 days, no strength data being obtained for other ages.

Crushing strength results for series A are also shown in table 5 and are plotted in figure 2. The use of Waylite dust as a replacement did not have the same beneficial effect upon the crushing strength of the concrete as is shown for flexural strength. For example, for the mixes in which the base water-cement ratio was 0.8 or higher, the strength of the concrete containing Waylite dust was in most cases slightly lower than the corresponding straight portland cement concrete, the percentage of reduction increasing as the amount of Waylite dust used as a replacement was increased. In the case of the group for which the base water-cement ratio was 0.70, the strength of the concrete containing the Waylite dust was approximately the same as that of the straight portland cement concrete.

DISCUSSION OF SERIES B

The results for series B are tabulated in table 6 and are shown graphically in figures 3 and 4. Figure 3 shows the flexural strengths and figure 4 shows crushing strengths. These data show, for both types of concrete (that is, the concrete containing natural sand and crushed limestone and the concrete containing Waylite aggregate), the flexural and crushing strengths after 28, 180, and 360 days' continuous moist curing as well as corresponding strengths after curing 180 days in moist air followed by 180 days in laboratory air.

The flexural strengths of the limestone concrete and the Waylite concrete can be compared by studying the individual panels of figure 3. For example, the second panel gives the flexural strength of the 6-sack limestone concrete, while the fourth panel gives the strength of

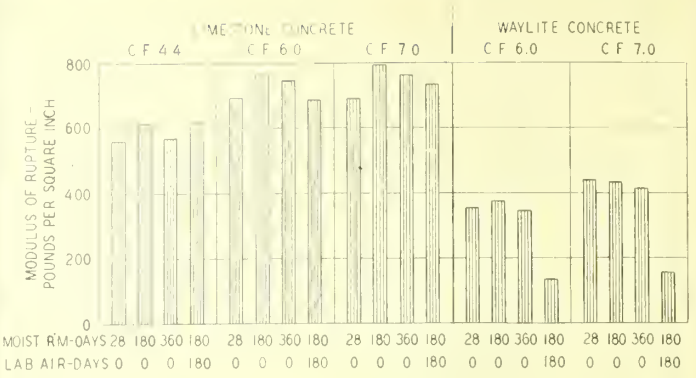

FIGURE 3.—EFFECT OF AGE AT TEST AND METHOD OF CURING ON THE FLEXURAL STRENGTH OF CONCRETE CONTAINING NATURAL AGGREGATES AND WAYLITE, SERIES B.

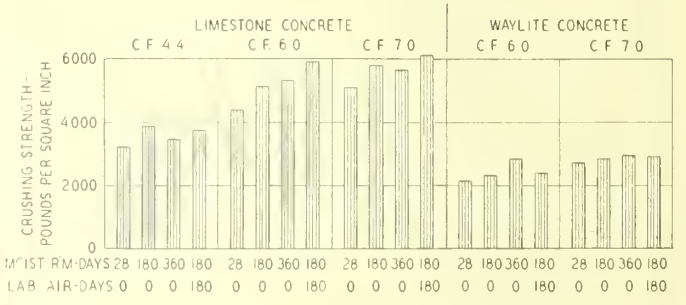

FIGURE 4.—EFFECT OF AGE AT TEST AND METHOD OF CURING ON THE CRUSHING STRENGTH OF CONCRETE CONTAINING NATURAL AGGREGATES AND WAYLITE, SERIES B.

the 6-sack Waylite concrete. The moist-cured concrete containing the Waylite aggregates developed only about one-half the strength of the limestone concrete. The same general trend applies to the 7-sack mixtures.

Comparing the two concretes on an approximately equal water-cement ratio basis (that is, 4.4-sack limestone concrete with 6-sack Waylite concrete), the Waylite concrete developed about 60 percent of the strength of the limestone concrete. These differences are no doubt due to the physical characteristics of the Waylite aggregate. Being of a cellular structure and very light in weight, it does not have the strength of a normal limestone such as was used in these tests as the basis of comparison. The strength of the concrete stored 360 days in moist air was higher than that of comparable concrete stored 180 days in moist air followed by 180 days in laboratory air. For the 6- and 7-sack limestone concrete, this decrease in strength averaged about 6 percent, while for the Waylite concrete the specimens cured in laboratory air for 180 days following 180 days in moist air and tested dry gave only about 40 percent of the strength of those cured wet for 360 days and tested wet.

The marked reduction in flexural strength obtained in these tests as a result of intermittent curing agrees very closely with results previously obtained in a series of tests of Haydite concrete.[1] In these tests Haydite specimens cured 180 days in the moist room followed by 180 days in air gave but 36 percent of the strength of similar specimens cured 360 days continuously wet. Specimens in the same series made of natural aggregates and cured under identical conditions gave 90

[1] The Effect of Curing Conditions on Strength of Concrete Test Specimens Containing Burnt Clay Aggregates, by W. F. Kellermann, PUBLIC ROADS, vol. 18, No. 3, May 1937.

TABLE 7.—*Results of sulphate exposure tests of series C* [1]

Mix No.	Waylite dust replacement by weight	Water-cement ratio, base mix	Weight loss after period of immersion indicated			
			26 months	39 months	52 months	98 months
	Percent		*Percent*	*Percent*	*Percent*	*Percent*
1 (a)	None	0.70	6.4	17.2	100.0	100.0
1 (b)	25		5.8	16.6	22.8	35.5
1 (c)	32		4.1	14.8	19.0	[2] 22.8
1 (d)	40		3.9	6.4	12.0	[2] 22.6
2 (a)	None	.80	21.8	100.0	100.0	100.0
2 (b)	25		13.2	25.2	34.8	100.0
2 (c)	32		12.0	17.6	27.4	62.4
2 (d)	40		6.8	17.6	25.1	35.4
3 (a)	None	.90	34.6	100.0	100.0	100.0
3 (b)	25		26.6	75.7	100.0	100.0
3 (c)	32		19.2	35.1	50.8	100.0
3 (d)	40		11.0	21.4	31.6	51.0
4 (a)	None	1.00	63.6	100.0	100.0	100.0
4 (b)	25		46.6	100.0	100.0	100.0
4 (c)	32		35.3	57.7	100.0	100.0
4 (d)	40		31.0	45.6	78.4	100.0

[1] Each value is the average of two tests except as noted.
[2] One test only.

percent of the strength of continuously moist-cured specimens.

In general, the data shown in figure 3 indicate that the strength of the Waylite concrete was about one-half of that obtained with comparable concrete (same cement content) made with natural aggregates when the test specimens were cured continuously moist. For specimens tested at 360 days, which were subjected to 180 days of moist curing followed by 180 days of air curing, the strength of the Waylite concrete was only about one-fifth of the strength of the concrete made with the natural materials. The limestone used in these tests is of a nature that tends to produce high-strength concrete, particularly in flexure.

Crushing strength data are shown in table 6 and plotted in figure 4, the arrangement being the same as in figure 3. For the continuously moist-cured specimens the results agree very closely with those obtained in flexure, the Waylite concrete giving about half the strength of the limestone concrete for equal cement content and about two-thirds for equal water-cement ratio. However, specimens cured moist for 180 days followed by 180 days' curing in air did not show the retrogression in strength that was found with the flexure specimens. This applies both to the Waylite concrete and to the limestone concrete. In fact, the limestone concrete subjected to the air curing gave about 10 percent higher strength than did the specimens cured continuously moist.

SERIES C WITH EXPOSURE TO SULPHATE ACTION

Periodic examination was made of the cube specimens from series A which had been immersed in sulphate solution, and the losses in weight, based on the wet weight of the specimens at the time of immersion, were determined. These losses, computed on a percentage basis, are given in table 7. They are also plotted in figure 5 in a manner to bring out the effect of richness of mix and also the effect of varying the quantity of Waylite dust used as a replacement. The water-cement ratios for the base mixes, mixes containing no Waylite dust, and the percentages of replacements were, of course, the same as in series A (see table 3).

The bar diagrams in figure 5, which extend to 100 percent, are broken for the reason that in no case did complete disintegration occur at exactly 39, 52, or 98 months but at some time between these periods and the time previous weight determinations were made.

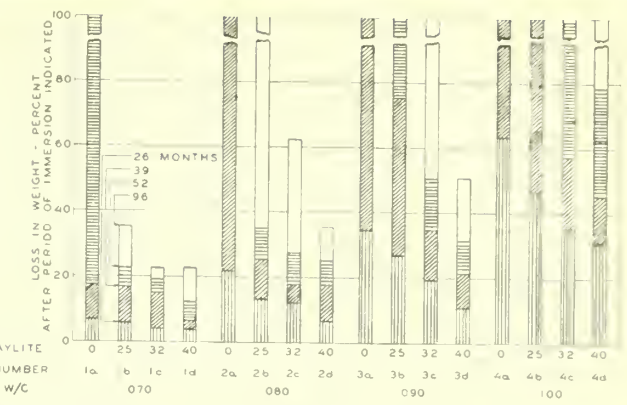

FIGURE 5.—EFFECT OF USING WAYLITE DUST AS A REPLACEMENT FOR PORTLAND CEMENT ON RESISTANCE OF CONCRETE TO ALKALI ACTION, SERIES C.

Considering the resistance of the base concrete as affected by water-cement ratio, it is evident that the durability was markedly decreased by increasing the water-cement ratio. The straight portland cement concrete showed losses after 26 months' immersion varying from 6.4 percent for a water-cement ratio of 0.70 to 63.6 percent for a water-cement ratio of 1.00. After 39 months' immersion, all of the plain concretes with water-cement ratios of 0.80 or higher had completely disintegrated while at 52 months the plain concrete with a water-cement ratio of 0.70 had completely disintegrated.

The use of Waylite dust as a replacement for a portion of the portland cement had a decidedly beneficial effect in these tests. Referring again to table 7 and to figure 5, it will be noted that, for any given base water-cement ratio, improvement in resistance to the action of the sulphate solution resulted from the use of the Waylite dust. For mixes 1 (a) to 1 (d), inclusive, the losses in weight after 26 months' immersion varied from 6.4 percent for the 0-percent replacement to 3.9 percent for the 40-percent replacement. This group had a low water-cement ratio of 0.70. In contrast, for the same immersion period, the group in which the base water-cement ratio was 1.00 showed losses ranging from 63.6 percent for the 0-percent replacement to 31.0 percent for the 40-percent replacement. The same general trends hold for the other immersion periods.

A study of the weight losses after over 8 years' (98 months) immersion furnishes interesting comparisons. In the lowest water-cement ratio group none of the mixes containing Waylite dust showed complete disintegration, the maximum loss in weight being 35.5 percent for the 25-percent replacement. In the next higher water-cement ratio group the mix containing 25 percent Waylite showed complete disintegration while the mix with 40 percent Waylite showed practically the same weight loss as that found with the 25-percent replacement in the group having a base water-cement ratio of 0.70, mix No. 1(b). In the group having 0.90 base water-cement ratio the weight loss of the mix containing the largest amount of Waylite was 51.0 percent with all other mixes showing complete disintegration. In the case of the group in which the base water-cement ratio was 1.00, all mixes showed complete disintegration at 98 months.

(*Continued on p. 250*)

TIRE WEAR AND COSTS

ABSTRACT AND CONCLUSIONS RESULTING FROM STUDIES AT IOWA ENGINEERING EXPERIMENT STATION

VALUABLE DATA ON TIRE PERFORMANCE are presented in bulletin 161, Tire Wear and Cost on Selected Roadway Surfaces, by R. A. Moyer, research professor of civil engineering, and Glen L. Tesdell, research assistant, of the Iowa Engineering Experiment Station at Ames, Iowa. The work reported was done in cooperation with the Public Roads Administration. The following is an abstract of the report and a full presentation of the summary and conclusions.

This study of automobile tires is part of a general investigation of vehicle operating costs, and of the roughness (in terms of riding comfort) and slipperiness of various types of roadway surface. The 10 sections of highway of various types, on which studies were made, varied in length from 20 miles to 320 miles. The sections or routes were located in four States—Iowa, Kansas, Missouri, and Wyoming—and were selected because they were considered as reasonably representative of the types of surface on the main highways of the United States. The scope of the investigation is indicated by the fact that it involved the use of eight automobiles with which detailed observations were made during 450,000 miles of carefully controlled driving.

Tires were the subject of special study because their cost is influenced to a greater extent by the type and condition of the roadway surface than is any other item of vehicle operating cost. The study provided data indicating the effect of each of 15 factors on the rate of tire wear, and of 12 additional factors contributing directly to tire carcass failure. It was determined that tire wear is influenced directly by: Car speed; rate of braking and accelerating; type of roadway surface; tire inflation; wheel alinement and wheel balance; atmospheric temperature; tire switching or "rotation" from one wheel position to another around the car; highway curves and grades; type, grade, and age of the tire; mechanical condition of the car; and the driving habits of the operator. Factors which contributed directly to tire carcass failures were: Neglected cuts, punctures, stone bruises, rim bruises, and fabric breaks; heat, fatigue, overload, and incorrect inflation; tread separation, deterioration of the rubber and fabric from oil and water, and damage from tire chains; and certain types of abnormal wear. The results of the tire wear measurements conducted by the Iowa State Highway Commission to determine the relative resistance to wear of various makes of 100-level and 90-level tires and of synthetic rubber tires supplement the data obtained in this investigation, thus making it possible to present a summary of all of the major factors that contribute to tire wear and tire failures. The term 100-level designates tires of the grade regularly used by car manufacturers as original equipment on new cars; the price of these tires is taken as a base (100 percent =100-level).

The studies indicated that, primarily, speed determined the rate of tire wear and that the 35-mile speed limit, imposed as a war measure in the interest of tire conservation, was based on sound premises. On the basis of the average wear rate observed on concrete pavements and bituminous surfaces in Iowa and Kansas, a life of 56,500 miles for automobile tires may be expected, with due attention to tire maintenance, if the car speed does not exceed 35 miles per hour; whereas at 65 miles per hour the life expectancy would be only about 18,700 miles.

It was found that variations in car speed affected tire wear to a greater extent than any other single factor. Stop-and-go studies, simulating car usage in cities, showed an extremely high rate of wear. It was found that when the wheels of a car are out of alinement the continuous braking effect which is created may cause tires to wear at a rate as high as 10 times the normal rate.

The investigation disclosed that there is a wide variation in tire costs on the various types of road surface, and a sharp rise in the costs as the number of stops per mile is increased. Improvements in tires and in the construction of highways have yielded big dividends to the motorist, but much improvement still is possible as is evident from the fact that the tire costs for travel on gravel surfaces are about double those for travel on concrete pavements.

The United States, in devoting its energies to the vigorous prosecution of World War II, has been confronted with a rubber shortage, and in consequence the supply of tires for civilian use has been sharply curtailed. The findings of this investigation, indicating so clearly the measures necessary for tire conservation, were summarized in nontechnical form in a pamphlet, Tire Wear and Tire Failures on Various Road Surfaces, published by the Public Roads Administration in 1943.

The results of the study show that highway engineers can aid in tire conservation by minimizing, within the limits of safety, the abrasiveness of surface treatments, and by keeping roads and streets in good repair to prevent tire bruises and punctures which may lead to blowouts. Careful attention by the vehicle owner to such measures as repairs of minor tire injuries, correct inflation, and elimination of exposure to oil and excessive sunlight or heat can contribute still more. These measures are of course equally efficacious in peacetime or in wartime. The report provides information that will be useful in highway planning and traffic control, and in the selection of maintenance methods and materials.

SUMMARY AND CONCLUSIONS

The significant data and conclusions of this investigation are as follows:

1. Results of tire wear measurements are reported for more than 2,000,000 miles of tire travel on selected sections of the major types of road surface in Iowa, Kansas, Missouri, and Wyoming.

2. Variations in car speed caused greater differences in the rate of tire wear (loss of tread rubber) than did variations in braking and acceleration, type and condition of road surface, tire inflation pressure, or any of the other factors investigated.

3. The rate of tire wear on concrete pavements at a nominal speed of 65 miles per hour was four times that at a nominal speed of 25 miles per hour.

4. The greatest change in tire wear with change in speed was observed when operating on the most abrasive surfaces, for which wear rates at 65 miles per hour

were in some cases as much as four times those at 35 miles per hour.

5. The least variation of wear with speed was observed when operating on bituminous surfaces sealed with thin coatings of asphalt or covered with moderately soft limestone chips held in place by asphalt. The rate of wear on these surfaces at 65 miles per hour was approximately double that at 35 miles per hour.

6. The higher rates of tire wear observed at the higher speeds were due to the greater amount of bouncing and slipping of the tires, the increased traction required at the rear wheels, and the increase in brake applications and braking time.

7. In stop-and-go tests, simulating city traffic conditions, the rate of wear was seven times that in normal driving on rural pavements at a speed of 25 miles per hour.

8. The average rate of wear of tires used regularly on city streets at the customary speeds was two to three times that of tires used on rural pavements at 45 miles per hour.

9. The rate of tire wear on dry pavements was approximately double that on the same pavements when wet.

10. When wheels are out of alinement, forward motion of the car produces a continuous braking effect which, in extreme cases, may result in tire wear at 10 times the normal rate.

11. Rate of tire wear during the first 500 miles of use was three times the rate after tires had been driven 3,000 to 6,000 miles.

12. Rate of tire wear was affected only slightly by variations in atmospheric temperature. On gravel surfaces the rate of wear was greatest in the summer months, but on dry concrete pavements and bituminous surfaces the rates of wear were greatest during the winter.

13. The rate of wear for the tires on rear wheels was from 130 to 200 percent of the rate on front wheels. As a general average, the tires on the right wheels wore 10 percent faster than those on the left wheels.

14. Switching or "rotating" all tires, including that on the spare wheel, from one wheel position to the next around the car every 2,000 miles is recommended as a means of equalizing tire wear.

15. Tire wear on curves at speeds that caused the tires to "scream" was more than 10 times the wear at speeds that did not result in sidewise skidding.

16. The rates of wear of tires of the 100-level grade made by various manufacturers, and normally supplied on new cars in prewar days, did not vary from the average by more than 10 percent.

17. The rate of wear of two synthetic rubber tires, on one car, exceeded the rate for the two regular 100-level tires by approximately 30 percent.

18. Increase in the rate of wear of tires, because of age, was less than 25 percent for ages up to 3 years. Effect of age on wear was twice as great for rear tires as for front tires.

19. The average life of 10 tires operated on concrete pavements was 36,651 miles. On gravel surfaces the average life for 20 tires was 23,160 miles. All of the test tires were used more miles per year than are those on the average passenger car and, therefore, under more favorable conditions as to age.

20. Records of car owners and of tire manufacturers indicate that in 1940 the average life of tires in the United States was approximately 22,000 miles.

21. The longest life of a 100-level tire in the wear tests made by the Iowa State Highway Commission was 70,000 miles, and the shortest life 20,000 miles. One of the synthetic rubber tires had a life of 36,000 miles.

22. Operation of a vehicle equipped with a set of tires with recapped treads made of 95 percent reclaimed rubber, indicated a life after recapping of 14,000 miles if speed were restricted to 35 miles per hour.

23. A life of 60,000 miles is a reasonable expectancy for 100-level tires with one recapping of reclaimed rubber, and restriction of speed to 35 miles per hour.

24. Many thousands of tire-miles are lost when tires become unserviceable because of avoidable carcass failures long before the treads are worn smooth. Among avoidable damages are abnormal tread wear; carcass weakness resulting from neglected cuts and punctures, stone bruises and fabric breaks, and loosening of inside cords by running on soft or flat tires; overload, incorrect inflation, and rim bruises; and tread separation, deterioration from oil, and long exposure to sunlight or heat.

25. In 132,000 miles of driving on gravel surfaces, there were 98 punctures as compared with a single puncture in the same distance on concrete pavements and one puncture in 72,000 miles on bituminous surfaces.

26. The heat, impact, and fatigue effects of operation on rough, loose, corrugated gravel surfaces resulted in tread cracking and blowouts. Approximately 30 percent of the original (new-to-smooth) tread-groove depth was still remaining when these tires became unserviceable because of carcass failures.

27. Continuous operation at 65 miles per hour on concrete pavements, with no change in atmospheric temperature, caused the pressure in 6.00–16 passenger car tires to increase 4.6 p. s. i. (pounds per square inch) in the rear tires and 3.5 p. s. i. in the front tires. For continuous operation under similar conditions the tire pressure was 1.5 p. s. i. higher at 65 miles per hour than at 35 miles per hour for both the front and rear tires.

28. The tire pressure rise when driving on gravel surfaces was 0.25 p. s. i. lower than that on concrete pavements for corresponding speeds and atmospheric temperatures.

29. An increase of 1.0 p. s. i. in tire pressure was observed for each 20° F. rise in atmospheric temperature.

30. A normal loss in tire pressure of 1 to 2 p. s. i. per week was observed during the winter, and of 2 to 4 p. s. i. during the summer.

31. An inflation pressure of 30 to 32 p. s. i. "cold" is recommended to obtain the greatest service from 6.00-16 tires.

32. The practice of decreasing the inflation pressure by letting air out of tires in hot weather is not recommended, unless pressures are checked and adjusted carefully and frequently.

33. The temperatures in the treads of 6.00–16 tires in continuous operation at 25 miles per hour on dry concrete pavements, averaged 35° F. higher than the atmospheric temperatures in summer and 55° F. higher in winter. When operating under similar conditions at 65 miles per hour the tread temperatures averaged 75° F. higher than the atmospheric temperatures in summer and 95° F. higher in winter.

34. Tread temperatures averaged about 40° F. lower on wet surfaces than on the same surfaces in the dry condition.

35. By making suitable allowance (an increase of 3 to 5 percent) for the changes in tire volume due to changes in tire pressure, the tire pressures for various tire-air temperatures can be computed by applying the laws relating to pressure and temperature of gases.

36. The costs for 6.00–16 tires and tubes operated on concrete pavements in Iowa and Kansas averaged 0.18 cent per car-mile; for bituminous surfaces in Kansas the average cost was 0.19 cent per car-mile; and for gravel surfaces in Iowa it was 0.37 cent per car-mile. The average speeds to which these costs apply were 42½, 43, and 37 miles per hour, respectively.

37. In 1910 the average life of tires was less than 3,000 miles and the average cost of a tire and tube was about $45. Thus, the average cost for tires and tubes in 1910 was about 6 cents per car-mile, or 33 times the average cost of the tires and tubes used on concrete pavements in this study.

38. The tire costs in the stop-and-go driving on concrete pavements averaged 0.61 cent per car-mile, or more than three times the corresponding cost for average cross-country driving.

39. Highway engineers can aid in tire conservation by minimizing the abrasiveness, within the limits of safety, when repairing or resurfacing roadways. Roads and streets should be kept in good repair, free from pieces of metal and glass that can puncture tires. Holes with sharp edges, which cause cuts and bruises that may result in blowouts, should be repaired promptly.

40. Compliance with restrictions of the wartime tire conservation program is necessary to keep civilian motor vehicles in operation until adequate supplies of new tires are available. The major items in the conservation program include limiting speed to 35 miles per hour, recapping of tires at the proper time, eliminating all nonessential travel, and periodic inspecting of tires. Other measures recommended to car owners are: Start and stop slowly, reduce speed on sharp curves and steep hills, check tire inflation each week, and keep loads within the rated capacity of the tires.

(*Continued from p. 247*)

CONCLUDING STATEMENT

These tests emphasize the importance of moisture condition at time of test upon the flexural strength of concrete containing aggregates of the type represented by Waylite. The results indicate that the moisture content of the concrete at the time of test has a far greater effect when aggregates of this type are employed than when normal aggregates are used and corroborate the results of previous investigations.

The use of Waylite as fine and coarse aggregate in concrete resulted in a marked decrease in both crushing and flexural strength as compared to the results obtained with concrete containing the natural aggregates used in these tests. The decrease was somewhat greater when the comparisons were made on an equal cement-factor basis than when based on a comparable water-cement ratio.

The use of Waylite dust as a replacement for a portion of the portland cement resulted in an increase in flexural strength and a slight decrease in crushing strength. Its use also markedly increased resistance to the destructive action of a sulphate solution consisting of 5 percent Na_2SO_4 and 5 percent $MgSO_4$. The tests emphasize the beneficial effect, from the standpoint of sulphate resistance as well as strength, of using a low water-cement ratio.

U. S. GOVERNMENT PRINTING OFFICE: 1946

COTTON FABRIC IN BITUMINOUS CONSTRUCTION

Reported by PAUL F. CRITZ, Senior Highway Engineer, Public Roads Administration

In the search for new markets for surpluses of cotton that accumulated some years ago, considerable interest was evinced regarding the feasibility of using cotton fabric in low-cost bituminous road surfacing. Opinions were advanced that use of the fabric might permit more economical construction and could result in more durable roads.

Scattered early road-building experiments with cotton fabric were uncontrolled and therefore inconclusive. Laboratory tests indicated that the material might be of some benefit, but the extent was undetermined. Beginning in 1936, extensive field tests were undertaken in the use of cotton fabric in bituminous surfacing, and 400 miles of experimental roads were built in 24 States. Investigation of this work, which in many cases was carefully controlled, revealed that the use of cotton fabric materially increased the cost of construction but generally produced no important benefits.

This article is published to make available a record of the details of these experiments and of the conclusions drawn from them.

THIS REPORT describes an investigation of the use of cotton fabric in bituminous highway construction and maintenance made cooperatively by a number of State highway departments and the Public Roads Administration. The investigation was prompted by the desire to improve the serviceability of low-cost bituminous surfaced roads by reducing costs, eliminating failures, and retarding deterioration. If the cotton fabric proved beneficial and economic in the reinforcement of bituminous surfaced roads, not only would improvement in service be obtained but an additional practical use would be found for a commodity of considerable importance to many States.

These objectives prompted early experimenting with cotton fabric in some of the Southern States. In 1926 an experiment was begun in South Carolina with fabric as a reinforcement for the bituminous surface treatments being constructed at that time and, by 1935, a number of experimental roads had been built which are described in some detail elsewhere.[1] Similar experimental roads were built in 1929 near Gonzales, Texas,[2] and in 1932 in Georgia [2] and Louisiana.[3]

The method of constructing these roads was similar in general to that used on many of the later experimental roads comprising the more extensive investigation described in this report. The road on which cotton fabric was to be used with a bituminous surface treatment was first cleaned of loose or foreign material and given a priming treatment of a bituminous material such as liquid asphalt, tar, or emulsion. Sometimes the cotton fabric was placed immediately after this application and sometimes it was placed after the priming material had penetrated into the nonbituminous base material and had dried.

The cotton fabric used was a fairly open, loosely woven cloth. Only a single thickness of cloth was used. In some cases a continuous strip of fabric was placed along each edge of the road and in others the full width of road being treated was covered with the fabric, using as many strips as necessary.

Following the placing of the fabric, alternate applications of bituminous materials and aggregate were spread and smoothed and the surface was then rolled. The types and quantities of materials used in these early experiments were those normally used by the particular State in the ordinary type of surface treatment. The use of the fabric had no appreciable effect upon the method of construction and none upon the appearance of the finished road surface. Only by markers placed for the purpose could the fabric sections be located after construction was completed.

These early experiments with cotton fabric in bituminous construction showed some promise but, because of the variable conditions under which the roads were built or resurfaced and the numerous types of fabric used, they did not offer a satisfactory basis for definite conclusions either as to the merits or limitations of cotton fabric or to the type of fabric best suited to the purpose. Data were not available to show whether the behavior of the experimental roads was due to the presence of the fabric or to other factors that have important effects on the behavior of bituminous-treated roads. Lack of information regarding the cost of construction and maintenance of these experimental roads made it impossible to determine whether apparent benefits were commensurate with the cost.

LABORATORY TESTS SHOWED FABRIC OF LITTLE VALUE

To provide desirable information on the merits and limitations of cotton fabric as a reinforcement for thin bituminous surfaces the Public Roads Administration conducted a laboratory investigation at the Arlington Experiment Farm in 1935.

This investigation included physical tests of bituminous mixtures with and without fabric and a service test of fabric in a normal bituminous surface treatment of the type for which the fabric was believed to be most practical. The physical tests of bituminous mixtures reinforced with and without fabric showed that the fabric neither added tensile strength to the mixtures nor reduced the tendency of the mixtures to crack under load. The behavior of the mixtures under test was practically the same whether or not fabric was used.

The service test was designed to simulate actual service conditions and was conducted on the indoor circular track which has been described in detail in PUBLIC ROADS.[4] The track was a circular trough 12 feet in diameter, 12½ inches deep, and 18 inches wide. A narrow inner trough served as a reservoir through which water could be introduced into the base and held at any desired level.

[1] Cotton-Fabric-Reinforced Roads by W. K. Beckham and W. H. Mills. Engineering News-Record, October 3, 1935, page 453.

[2] Cotton Fabric for Reinforcing Bituminous Surfaces on Highways, Highway Research Abstracts, No. 32, July 1936, p. 4.

[3] Cotton Cloth Used to Waterproof Road Surface by Arnold Davis. Engineering News-Record, August 18, 1932, page 194.

[4] Circular Track Tests on Low-Cost Bituminous Mixtures, by C. A. Carpenter and J. F. Goode, PUBLIC ROADS, vol. 17, no. 4, June 1936.

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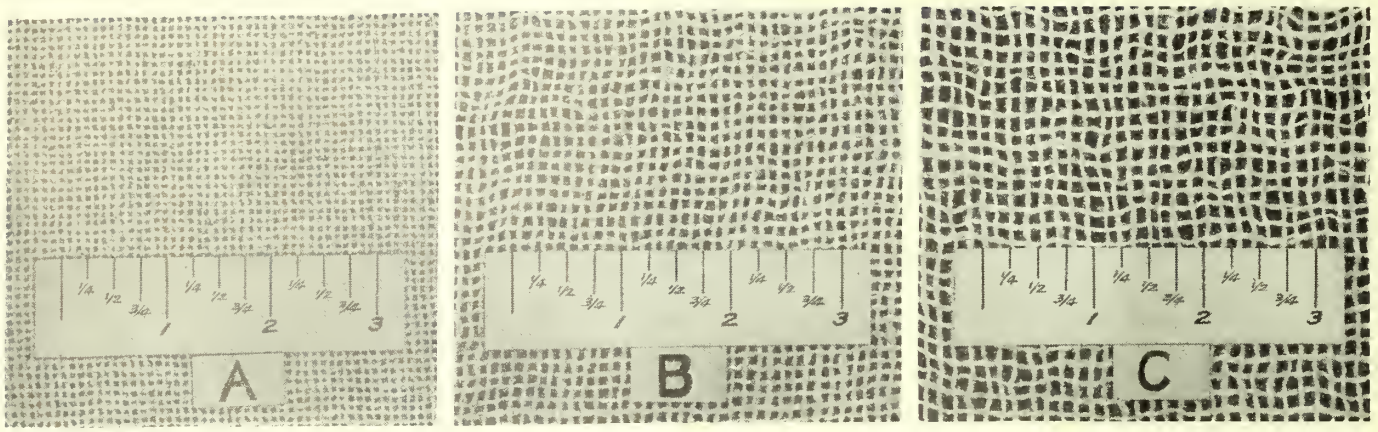

FIGURE 1.—THE TYPES OF COTTON FABRIC A, B, AND C USED IN BITUMINOUS CONSTRUCTION HAD 12, 9, AND 7 THREADS PER INCH AND WEIGHED 5.30, 4.25, AND 3.20 OUNCES PER SQUARE YARD, RESPECTIVELY. THE COST WAS 7.6, 6.1, AND 4.6 CENTS PER SQUARE YARD OF ROAD SURFACE F. O. B. DESTINATION.

Two automobile wheels, equipped with 6.00 by 20 low pressure tires inflated to 35 pounds pressure and mounted on the ends of a centrally pivoted steel beam, were used for compacting the base and surface courses and for testing the wearing surface. The entire weight of the wheel and beam assembly, which was carried by the test surface, was 1,600 pounds or 800 pounds per wheel. This test load was later increased to 1,000 pounds per wheel.

A bituminous surface treatment with a supporting base of sand-clay-gravel was selected for the test. Two classes of base were used: One was composed of materials that met the requirements for satisfactory base-course materials; the other was composed of materials definitely inferior in quality. On each of these bases, two sections of bituminous surface treatment were constructed which were identical except that one section contained cotton fabric and the other did not.

The sand-clay-gravel bases were thoroughly compacted and then primed with one-fourth gallon of tar per square yard. The fabric used in the two reinforced sections was placed in the tar prime while it was still wet. Twenty-four hours later 0.2 gallon of hot tar per square yard was spread on the surface, covered with 35 pounds per square yard of crushed limestone and then lightly rolled. Following this operation 0.3 gallon of hot tar per square yard was spread and covered with 10½ pounds of ⅜-inch crushed limestone and 4½ pounds of coarse sand. Thorough rolling completed the surface treatment.

The test consisted of subjecting the surface to traffic simulated by the automobile wheels. No apparent change occurred in the surface until water was admitted to the base course, after which failures began to develop. Testing continued until practical destruction of the surfaces had occurred. The results of this test may be summarized briefly as follows:

1. On the satisfactory base there was no significant difference between the section that contained fabric and the one that did not.

2. On the base of inferior quality the section containing fabric remained in a satisfactory condition under traffic for a period approximately 25 percent longer than the section without fabric.

3. The test indicated that the fabric might be of some value under certain conditions but the extent of benefit could not be determined from this test.

At the conclusion of the test run, mat samples were removed from the sections laid on both types of bases and the fabric contained in them was recovered. In both instances the fabric had been nearly destroyed.

SEARCH FOR NEW USES OF COTTON AN INCENTIVE TO FURTHER EXPERIMENT

Because of the great surplus of cotton that had accumulated, Congress in 1936 authorized the Department of Agriculture to purchase large quantities of cotton for the purpose of promoting and encouraging its use in new fields. Considerable interest developed regarding the feasibility of using cotton fabric in low-cost types of bituminous construction. Opinions were advanced that fabric would act as a reinforcement for holding intact the edges which are the weakest points in thin bituminous surfaces; that the fabric, when impregnated with tar or asphalt, would serve as a membrane which would waterproof the road base and thereby lessen or eliminate the destructive effects of moisture; that less expensive base construction would be required; and that a practical use could be made of certain grades of cotton staple for which the demand was not great.

It was believed that by using some of this cotton in highway construction under a variety of known conditions, sufficient data might be accumulated to provide definite information as to its value and limitations. Accordingly, the Department of Agriculture, after preliminary laboratory study and experimenting, prepared specifications for three types of fabric believed suitable for an experimental program. These specifications are given in the appendix (*page 274*). The fabrics produced under them are shown in figure 1.

The fabric was furnished without charge by the Department to the State highway departments on the condition that it be used in highways constructed in an approved manner and that reasonable care be taken in obtaining data on the service behavior and cost of maintaining the roads in which it was used so that evaluation might be made of the benefits derived.

EXTENSIVE FIELD TESTS WERE UNDERTAKEN

Twenty-four States requested approximately 6,000,000 square yards of fabric; enough to surface more than 500 miles of 20-foot roadway. Although the original specifications provided for three widths of

TABLE 1.—*Extent of experimental use of cotton fabric*

State	Fabric supplied	Sections reported	Fabric reported used [1]				Lengths of road in which fabric reinforcement was used			
			Type A	Type B	Type C	Total	Type A	Type B	Type C	Total
	Square yard	*Number*	*Square yard*	*Square yard*	*Square yard*	*Square yard*	*Miles*	*Miles*	*Miles*	*Miles*
Alabama	1,250,650	18	361,013	367,318	327,968	1,056,299	30.02	30.54	27.27	87.83
Arizona	115,473	[2] 3	1,993		1,806	3,799	.12		.15	.27
Arkansas	122,996	2	30,189	31,958	30,191	92,338	2.51	2.66	2.51	7.68
California	66,000	6	5,312	13,593	12,885	31,790	.40	1.30	.98	2.41
Florida	54,118	1	17,402	17,720	17,752	52,874	1.45	1.47	1.47	4.39
Georgia	144,321	3	36,079	36,732	36,079	108,890	3.00	3.05	3.00	9.05
Indiana	113,957	2	37,986	37,986	37,986	113,958	3.50	3.50	3.50	10.50
Maine	18,186	1	6,050	5,626	5,809	17,485	.50	.46	.48	1.44
Massachusetts	43,412	3	15,990	15,340	10,559	41,889	1.47	1.41	.97	3.85
Michigan	447,975	9	117,413	120,275	113,715	351,403	8.90	9.11	8.62	26.63
Mississippi	109,330	3	11,042	23,889		34,931	.92	1.98		2.90
Missouri	257,400	6	87,033	76,970	79,672	243,675	6.60	5.83	6.03	18.46
Montana	39,600	1	13,200	13,200	13,200	39,600	1.00	1.00	1.00	3.00
Nevada	66,116	1	13,200	13,200	13,200	39,600	1.00	1.00	1.02	3.02
New Hampshire	21,706	1	7,017	7,297	6,886	21,200	.64	.67	.63	1.94
New Jersey	102,497	5	31,949	36,969	29,874	98,792	2.62	3.14	2.49	8.25
New York	842,097	21	217,405	230,801	238,040	686,246	18.53	20.03	20.70	59.26
North Carolina	1,115,178	17	314,563	314,765	313,784	943,112	27.41	27.48	27.38	82.30
Oregon	36,166	1	11,518	11,396	11,518	34,432	.95	.94	.95	2.84
Rhode Island	125,096	[3] 2	2,676	2,676	4,334	9,686	.05	.05	.05	.15
South Carolina	669,951	18	210,611	206,302	208,691	625,604	16.57	16.24	16.34	49.15
Tennessee	72,089	1	13,844	23,836	14,531	52,214	1.15	1.98	1.21	4.34
Virginia	89,482	7	24,038	34,455	22,942	81,435	1.79	2.68	1.72	6.19
Washington	117,006	7	30,078	25,161	25,528	80,767	2.48	2.07	2.11	6.66
Total	6,040,802	[4] 139	1,617,601	1,667,465	1,576,953	4,862,019	133.56	138.27	130.68	402.51

[1] When amounts of each type used were not specifically stated, it was assumed that equal amounts of each type were used.
[2] Types and amounts not given for two sections. [3] Includes one road section only. No other information reported.
[4] Two types of construction were employed on each of two sections.

material, the fabric was actually furnished in widths ranging from 56 to 90 inches, depending upon the width of the road to be surfaced. In order to provide information on the type of fabric best suited for the purpose, each width was furnished in approximately equal quantities of the three weights and weaves. Type A weighed 5.30 ounces per square yard and had a thread count of 12 per inch in each direction. Type B weighed 4.25 ounces per square yard and had a thread count of 9 per inch. Type C weighed 3.20 ounces and had a thread count of 7 per inch.

Taking into account the overlapping of adjacent strips of fabric, the average cost per square yard of road surface of the three types of fabric at the points of delivery was: Type A, 7.6 cents; type B, 6.1 cents; and type C, 4.6 cents. The average of the reported costs of handling on the job was 1.25 cents per square yard.

Experimental sections were constructed in each of the 24 States that requested fabric and approximately 80 percent of the fabric furnished was reported as having been placed in service. The amount of fabric furnished, the number of sections constructed, and the amount of fabric used therein are shown for each State in table 1. The types of construction used were as follows:

Type of construction	*Number of sections reported*
Surface treatment	99
Plant mix	15
Road mix	12
Seal treatment on road mix	2
Surfacing wooden bridge floors	10
Bituminous shoulder treatment	2
Base reinforcement	1
Total	141

USE OF FABRIC DID NOT COMPLICATE CONSTRUCTION

Practically all types of low-cost bituminous construction and re-treatments are represented by these experimental sections, which were built in all parts of the country. Variations in temperature and moisture conditions likely to be found in any State are believed to be represented. The types of soil and character of bases upon which the experimental surfaces were built, as well as the types of bituminous materials and aggregates used, also include many kinds, so that in general any set of conditions normally encountered in this country was represented in the experiments.

The use of fabric did not involve any serious construction difficulties despite the fact that in practically all instances contractors, maintenance crews, and engineers alike were without experience in its use in bituminous construction and there was little information available to guide them. Many questions were raised relative to the procedure best suited to insure success of the experiments. Chief among these were: The allowance to be made for absorption of bitumen by the fabric; the relative merits of asphalt and tar for impregnating the fabric; the time of placing the fabric, that is, in the fresh prime or after the priming application had dried; satisfactory methods of holding the fabric in place; and protection of the fabric in place during subsequent construction operations. Since specific information on these matters was not available, the rules of common sense based upon general construction experience were followed.

Allowance for absorption varied from practically none to 0.05 gallon of material per square yard, depending upon the weight of the fabric. Both tar and asphalt were used as priming materials. Fabric was placed in the fresh prime on many experimental sections and on the dried primed base on many others. Various methods of holding the fabric in place were reported to have been used. When the fabric was placed in the wet prime it was sometimes fastened and sometimes it was not. Fastening devices included roofing nails and common fence staples, and were spaced at intervals ranging from 3 feet to 50 feet. They were placed only on the outside edges of the outer strips in some instances and in other instances both edges of all strips were fastened. When laid on the dried primed base, some type of fastener had to be

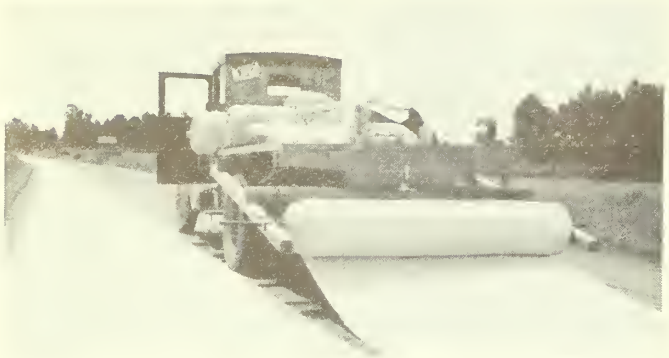

FIGURE 2.—LAYING COTTON FABRIC WITH A TRUCK-MOUNTED REEL.

used to prevent displacement of the fabric by the wind. Methods reported included use of roofing nails, staples, and small amounts of bituminous material spotted at intervals along the edges. In some instances small amounts of sand or fine stone or of bituminous mixture were spread on the fabric prior to surfacing operations but in most instances the fabric was not covered except by the normal operation that followed.

The wide range in reported costs of handling and placing the fabric includes actual costs accurately kept in some instances as well as contractors' bid prices. Having no experience as a guide to bidding on such items the contractors naturally set prices that could be expected to insure no loss on that item. However, on one of four special cooperative experimental sections where the work was done by State forces all items chargeable to actual handling and laying of the fabric totaled 0.16 cents per square yard. This particular road was built in a State where considerable early experimenting had been done and where some general information was available.

FABRIC USED IN NEW CONSTRUCTION AND IN RESURFACING

Fabric was used both in new construction and in the re-treatment, or resurfacing, of old bituminous surfaces. In new construction, where a wearing surface was laid on a nonbituminous base composed of such materials as topsoil, sand-clay, clay-gravel, stone, or chert, the base was first primed with a liquid bituminous material such as tar or asphalt. The fabric was then placed on the primed base and the bituminous surface constructed in the usual manner.

Where the bituminous surface mat was to be constructed of plant-mixed material, the fabric was generally given a tack coat of liquid bituminous material prior to placing the surface mixture. Where the surface mat was to be constructed by making alternate applications of bituminous material and aggregates, the fabric was impregnated by the first application of bitumen.

In re-treatment work where a new surface was being built on an old bituminous surface, the fabric was laid directly on the old surface and then given a tack coat of bituminous material, after which the new surface was constructed in the usual manner.

The fabric, which came in rolls, was rolled out over the surface in a longitudinal direction with adjoining strips overlapping 2 or 3 inches. Three or more strips were ordinarily used, the number depending upon the width of the surface, the width of the fabric used, and whether full width or half width construction was employed. After placing, the fabric was smoothed to remove wrinkles.

Placing of the fabric was done in various ways. Sometimes the rolls were laid on the base and rolled forward by hand. Sometimes they were suspended from the rear end of a truck and the fabric unrolled as the truck moved forward. In another method the roll was suspended on a pipe carried by two men or supported by a pair of wheels. The method used on a particular job depended upon the men and equipment available and upon the preference of the engineer in charge. Two of the methods used are shown in the cover illustration and figure 2.

On new construction, the fabric was sometimes laid immediately after the prime application so that it would be impregnated with the priming material and would be held firmly in place as the latter dried and hardened. Sometimes it was laid after the priming material had penetrated and dried thoroughly and was then impregnated with the subsequent applications of bituminous material. Figure 3, A shows the fabric laid in the wet prime and figure 3, B shows the fabric laid on the dried primed base. In both instances the fabric was ready for the subsequent applications of bitumen and aggregate or plant-mixed material.

Various methods were used to prevent displacement of fabric by the wheels of the distributors while spreading bituminous material. Sometimes a small amount of aggregate was spread first and sometimes the distributor wheels were wiped with a mixture of kerosene and light lubricating oil. In many instances, however, satisfactory results were obtained simply by keeping the distributor wheels clean and dry. As long as the wheels did not come in contact with the bituminous materials being applied, no difficulty was encountered because of fabric being displaced. Figure 4, A shows the distributor wheels being oiled just prior to applying hot asphalt on the impregnated fabric. Figure 4, B

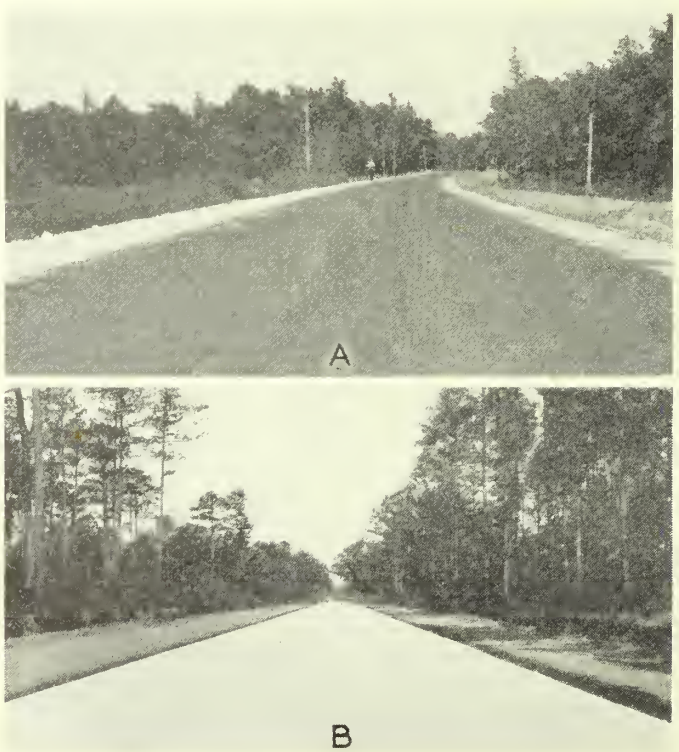

FIGURE 3.—FABRIC WAS IMPREGNATED BY LAYING IT IN WET PRIME (A) OR WAS PLACED ON A DRIED PRIMED BASE (B) BEFORE APPLYING HOT BITUMINOUS MATERIAL.

shows the distributor applying the hot asphalt. On this section no special precautions were taken, except to keep the wheels clean, and no trouble was met.

FOUR SPECIAL SECTIONS INCLUDED IN THE EXPERIMENT

Among the many sections constructed with cotton fabric were four special experimental roads in which the Public Roads Administration actively cooperated with the State highway departments of Alabama, North Carolina, South Carolina, and Tennessee. In these special experiments the roads were constructed in accordance with the standard practice of the State in which built, but more information regarding them was obtained than is ordinarily deemed necessary in routine construction. An examination of the base and subgrade materials was made prior to construction so that sections containing the different weights of fabric and one section without fabric could be constructed on bases having the same characteristics in order that a proper evaluation of future service behavior could be made. On each experimental road the bituminous construction was the same for all sections except that a slightly greater amount of bituminous material was used on the fabric sections to compensate for the absorption of the bitumen by the fabric.

Construction of the Alabama cooperative experimental road was completed in November 1936. About 6 miles of road were reinforced with fabric. The bituminous mat was constructed by the double surface treatment method. The soil base was chert. On this road each of the three types of fabric was laid in two ways: (*a*) Immediately after the tar prime had been applied and (*b*) after the prime had dried and hardened. The bituminous binder was hot asphalt for the first application and rapid-curing cut-back asphalt for the seal coat applications. The cover was crushed slag.

The North Carolina cooperative experimental road was completed in June 1937. It was 14½ miles long and contained approximately 8 miles of road reinforced with fabric. The base was composed of sand-clay mixtures that varied in plasticity. Construction was controlled to provide four groups of sections whose plasticity indexes were nonplastic to 4, 4 to 8, 6 to 10, and 12 to 16, respectively. In each group, sections were constructed with and without fabric. The bituminous construction was very similar to that employed on the Alabama experimental road except that the fabric was laid only after the primed base was dry. The cover aggregate was crushed stone.

The South Carolina cooperative experimental road was completed in September 1937. It was 6.7 miles in length and contained approximately 3 miles of road reinforced with fabric. The base was topsoil and was very uniform in its soil characteristics except as to the grading of the topsoil particles. On this road, eight sections were constructed, four containing fabric and four without fabric. Bituminous construction was approximately the same as that used on the North Carolina experimental road except that the liquid bituminous materials used in the seal treatment were a rapid-curing cut-back asphalt on some sections and a quick-breaking asphaltic emulsion on others.

The Tennessee experimental road was completed in August 1937. It was about 8 miles in length and contained approximately 3 miles of road reinforced with fabric. The base on most of the road was cherty clay, stabilized under traffic with crushed stone. On one short portion a waterbound macadam base was built.

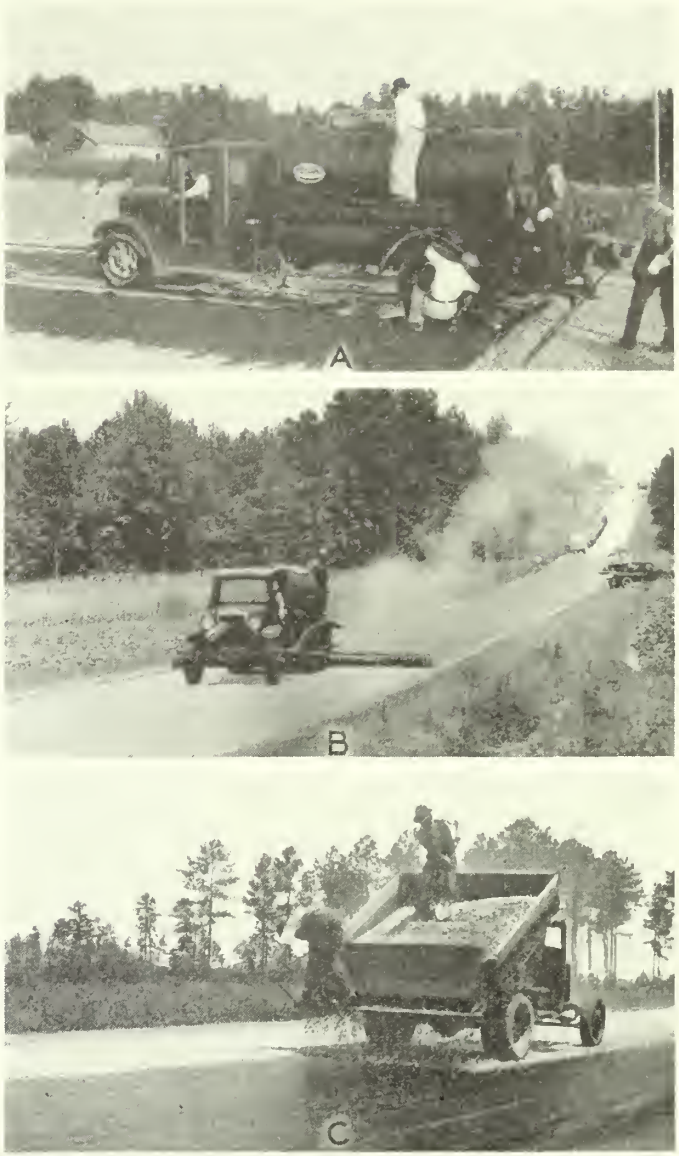

FIGURE 4.—(A) OILING THE DISTRIBUTOR WHEELS PREVENTED DISTURBING IMPREGNATED FABRIC. PAPER STRIPS UNDER THE WHEELS KEPT OIL FROM DRIPPING. (B) APPLYING HOT ASPHALT BINDER ON FABRIC LAID ON DRIED PRIMED BASE. (C) TRUCKS IMMEDIATELY FOLLOWING THE DISTRIBUTOR BACKED UP AND SPREAD THE COVER STONE UNIFORMLY.

Two types of bituminous contruction were employed. One was the surface-treatment type similar to that used in the other three States, where the fabric was laid on the primed soil base. The other was the road-mix type in which the bitumen and aggregate were mixed on the road, leveled, and compacted by rolling. After a short period of time under traffic, a seal treatment was applied and on this part of the experimental road the fabric was laid on top of the road-mixed mat immediately before applying the seal treatment.

Three of these experimental roads were built by contract and one by State forces. The methods used in each were those required by the State standard specifications and were essentially the same as those used throughout the country for similar types of contruction.

METHOD OF PLACING SURFACE TREATMENT DESCRIBED

The method of placing the surface treatment was very similar on all four experimental roads and was done

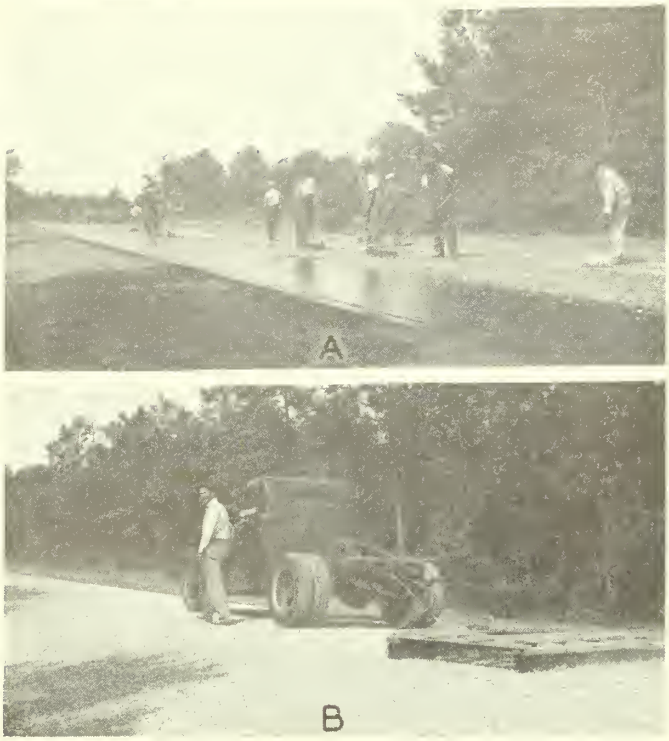

FIGURE 5.—(A) INITIAL HAND-BROOMING INSURED UNIFORM AGGREGATE COVERAGE. (B) A FIBER BROOM DRAG COMPLETED THE SPREADING WHICH WAS FOLLOWED BY MODERATE ROLLING.

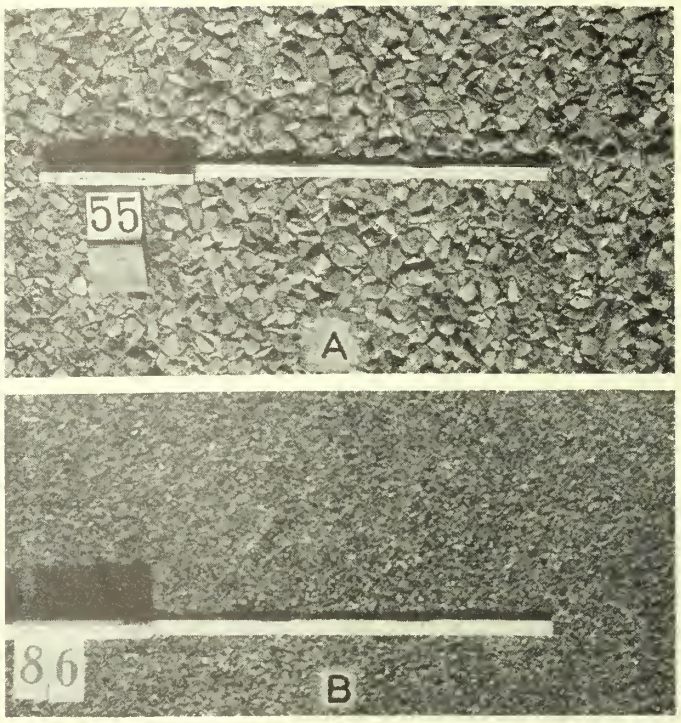

FIGURE 6.—(A) A TYPICAL SURFACE AFTER 20 DAYS OF TRAFFIC PRIOR TO SEAL COAT APPLICATION. (B) A WELL-SEALED SURFACE TREATMENT. USE OF FABRIC IN THESE SURFACES HAD NO EFFECT ON THEIR APPEARANCE.

essentially as follows: After the fabric had been placed as previously described, hot bituminous material was applied directly on the fabric. Immediately following this application, crushed stone or slag was spread from trucks provided with tail-gate spreaders, care being taken to keep the truck wheels always on the cover material. This operation is illustrated in figure 4, C. To insure uniform coverage, the trucks were all loaded uniformly either by using a strike-off board or by weighing each load, and each load was spread over a measured distance. When the cover material had been spread from the trucks it was lightly broomed by hand and with a fiber broom drag as shown in figure 5, A and B. When the cover had been spread uniformly so that no uncovered area or areas containing an excess of material remained, the surface was rolled and immediately opened to traffic for a short period during which time hand-spotting, brooming, and rolling were continued intermittently to insure seating all the cover.

A seal coat was applied either immediately or after the treatment had been subjected to traffic for periods not exceeding 3 weeks. The seal coat consisted of two applications of a liquid bituminous material and one spread of ⅜-inch maximum size aggregate. The total amount of liquid bituminous material used was approximately the same as that of the hot-application material but the fine aggregate used was about half the amount of the coarser aggregate used with the hot-application material. The fine aggregate was spread after the first application of liquid bitumen and with the same uniformity and care used with the coarser aggregate. It was either broomed or broomed and rolled before the second application of liquid bitumen was made, depending upon whether the seal stone was to be penetrated by the application or mixed with it. Sealing in this manner adds practically no thickness to the mat, as the small particles fill the interstices in the coarser stone. The small stones also reduce the rough texture of the surface and give it a pleasing appearance. Because of the relative richness of the bituminous material, the seal coat provides a waterproof surface. The presence of fabric in this type of construction influences neither the type of seal coat used nor the methods of constructing it. Figure 6, A is a typical view of a completed surface before the seal treatment was applied and figure 6, B is a typical view of a well-sealed surface.

SPECIAL EXPERIMENTAL ROADS WERE INSPECTED IN DETAIL

A detailed inspection of these four special experimental roads was made in October 1938 by the author. On three of them, samples of the bituminous mats were taken and the fabric was recovered for examination. In all, 14 samples were taken, 2 of which were cut from areas where softening of the soil base had caused the bituminous mat to settle and crack. The remaining 12 samples were taken from areas that were in excellent condition and that had neither required nor received any maintenance since construction. No evidence of movement of the base or of the surface was apparent in the good areas.

Without exception the fabric in the samples taken was in an unsatisfactory condition and in many instances it was practically destroyed. On these sections it not only had not prevented failures caused by inferior base material but it did not remain in good condition even where base and surface failures had not occurred. Figure 7, A shows the only unsatisfactory area on one of the cooperative experimental roads some 18 months after construction. The type A fabric removed from this area is shown in figure 7, B and as might be expected its condition was very poor. Figure 7, C is a general view of a section of another co-

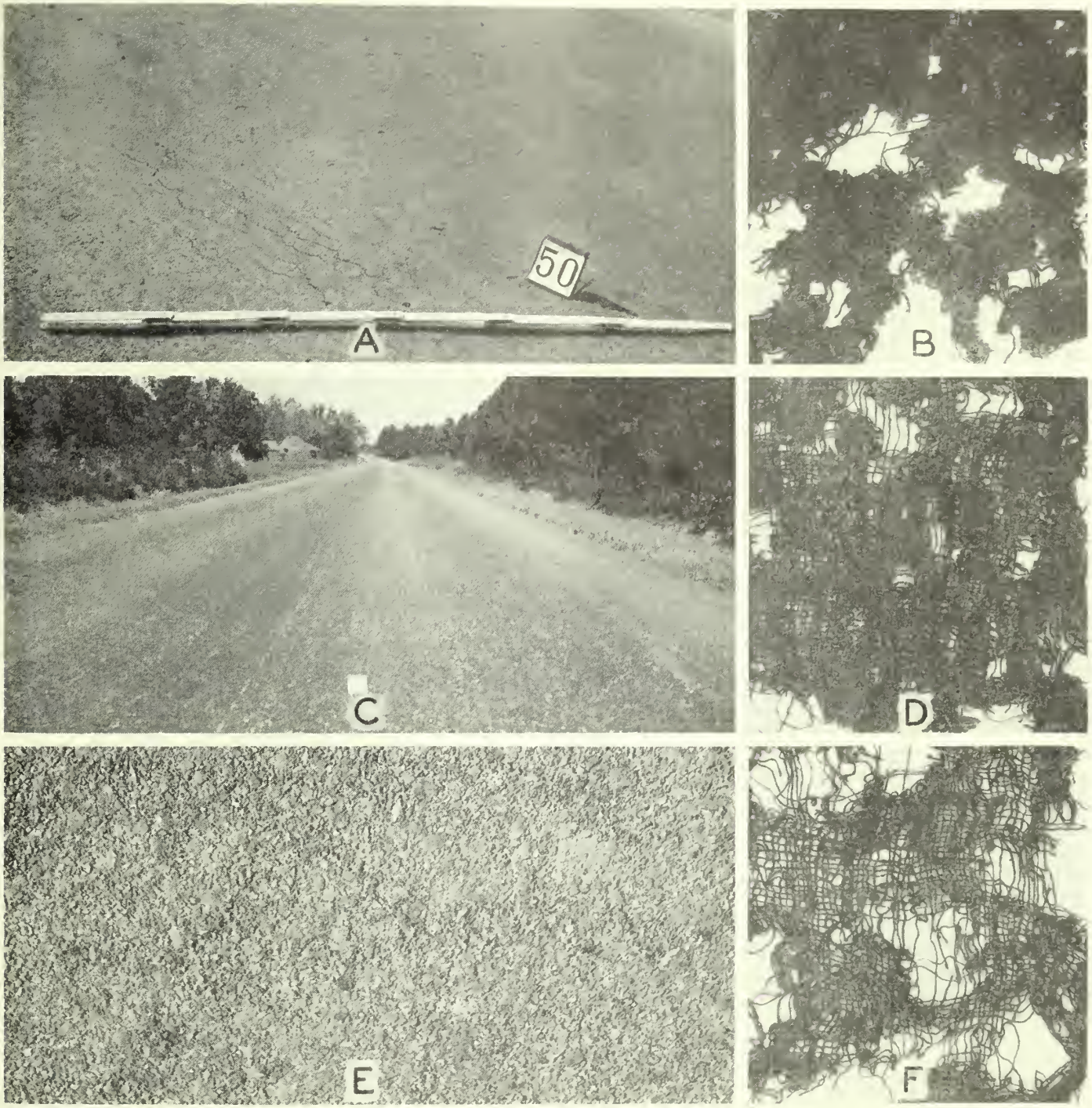

FIGURE 7.—RECOVERED FABRIC WAS FOUND BADLY DETERIORATED AND INCAPABLE OF AFFECTING FUTURE BEHAVIOR OF THE ROAD. (A) FAILURE OF THIS ROAD DUE TO A POOR BASE WAS NOT PREVENTED BY THE USE OF TYPE A FABRIC. (B) THE MATERIAL RECOVERED WAS IN POOR CONDITION. (C) THIS ROAD WAS IN EXCELLENT CONDITION AFTER A YEAR OF SERVICE EVEN THOUGH THE TYPE B FABRIC USED IN IT HAD PARTIALLY DISINTEGRATED (D). THE FINE APPEARING SURFACE (E) ADJOINED THAT SHOWN IN (C). THE TYPE C FABRIC RECOVERED FROM IT WAS NEARLY DESTROYED (F).

operative experimental road which was in excellent condition throughout its length 1 year after construction. The type B fabric removed from the area shown was not in good condition as is shown by figure 7, D. Similarly, figure 7, E shows a nearby area in excellent condition but the type C fabric removed from it was practically destroyed as shown by figure 7, F.

MANY OTHER FABRIC-REINFORCED ROADS WERE EXAMINED

In addition to these four cooperative experimental roads 80 other sections containing cotton fabric were inspected by the author in the fall of 1938 and the spring and summer of 1939. These sections were located in 16 States and contained more than 225 miles of road reinforced with fabric. The fabric was used in new construction and in the re-treatment of old bituminous surfaces. In both cases the new mat was constructed either by the surface-treatment method, the road-mix method, or by using plant-mixed material.

The use of the surface-treatment method, since it involved no manipulation other than light brooming and rolling, presented no special difficulty or change in

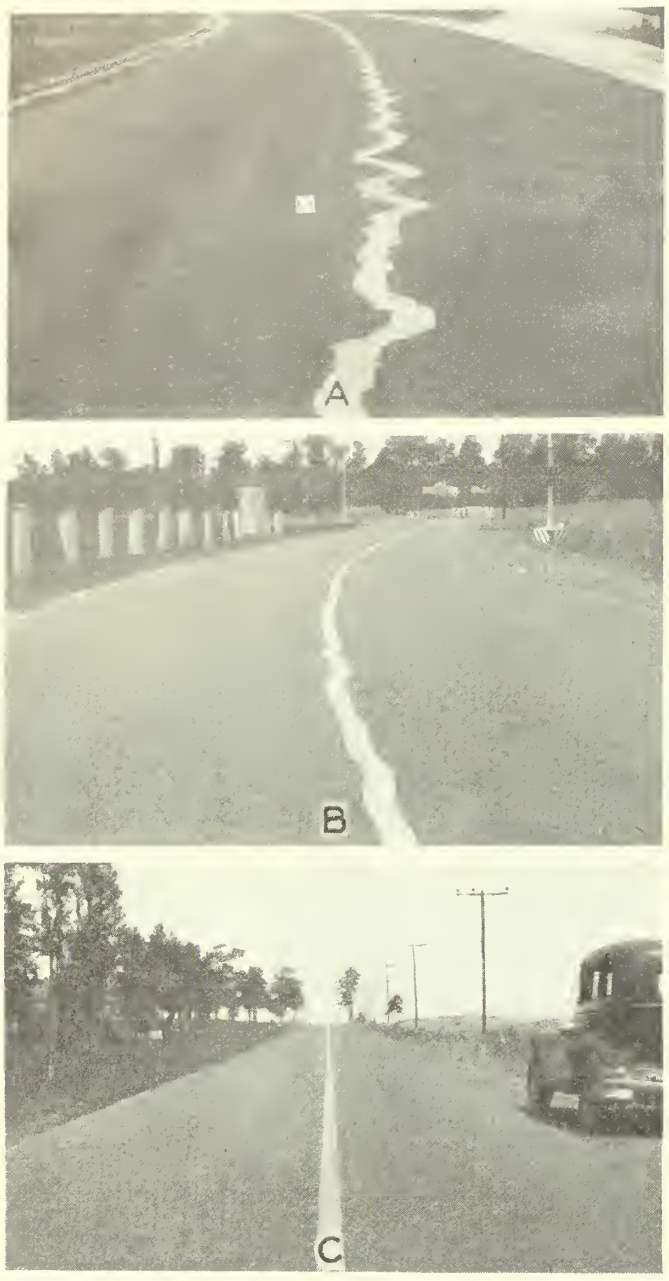

FIGURE 8.—WHERE FABRIC WAS USED IN THE RE-TREATMENT OF AN OLD SURFACE, THE NEW MAT SOMETIMES SLIPPED LATERALLY. MOVEMENT, AS SHOWN BY THE CENTER STRIPE, WAS GREATER IN THE SURFACE CONTAINING TYPE A FABRIC (A) AND WAS LESS IN THE SURFACE CONTAINING TYPE C FABRIC (B). NO SLIPPAGE OCCURRED WHERE FABRIC WAS NOT USED (C).

method of operation on sections that contained fabric.

Road mixing might not appear practicable where fabric was being used but one State found this method to be very satisfactory in re-treatment work provided sand was spread on the fabric immediately after it had received an application of bitumen. Additional aggregate and bitumen were then spread and road mixed. The thin blanket of impregnated sand provided a protective cover for the fabric during the mixing operation which, however, had to be carried on rather carefully. It was anticipated that any sand not held by the initial application of bitumen either would be incorporated in the mix or would be impregnated by the liquid bitumen used in the mix, so that no uncoated sand would be left between the fabric and the mixed material.

A number of the sections inspected were constructed in this manner and, in general, they were in excellent condition. No difference between the sections with and without fabric could be seen. On only one project were failures observed that apparently were not attributable to inadequate base support. In this instance pot holing had occurred on the sections where fabric was used and had extended through the mix proper, exposing a thin layer of bitumen-coated sand above the fabric. Here, apparently, a layer of uncoated sand had prevented the road mix from bonding to the old surface and to the fabric.

When plant-mixed material was used in constructing a new surface or re-treating an old one, some difficulty was reported in the construction operation where spreader boxes were pulled on skids or where mechanical finishers were used, unless the fabric was protected by first spreading a small amount of the prepared mixture on it. On the sections that did not contain fabric such precautions were unnecessary, of course.

Regardless of the type of construction, the failures that occurred were as prevalent on the fabric sections as on those without it. In addition, one type of failure seen only on sections where a surface treatment mat had been laid on an old bituminous surface was the tendency displayed by the mat containing fabric to slip. On one road the slippage was greatest on the type A fabric sections, less pronounced on the type B fabric sections, and least noticeable on the type C fabric sections. No slipping had occurred on the sections that did not contain fabric. Figure 8, A, B, and C illustrates this rather unexpected behavior.

FABRIC USED ON WOODEN BRIDGE FLOORS

In a number of States fabric was used in the bituminous surfacing of wooden bridge floors. Reports indicate that satisfactory results were obtained where the bridge deck was rigid. Ordinarily, when used on wooden floors the fabric was tacked to the deck and the bituminous mat was then constructed in the usual way.

It was reported that the fabric did not necessarily prevent fine hair cracks from forming over the spaces between the planks but that it did prevent raveling of the mat at the edges of such cracks and assisted in holding the mat together, thereby making a waterproof covering for the wooden floor and reducing its tendency to decay. On two adjoining wooden bridges in South Carolina that carry the same traffic and are identical in construction, fabric was laid on the floor of one bridge and not on the other immediately prior to constructing a bituminous surface mat. The mat containing the fabric had remained in excellent condition up to the time of the inspection. It furnished a waterproof covering for the untreated timber floor and stringers which likewise remained in good condition. Although the green planks with which the floor was laid had shrunk on drying and gaps up to one-half inch in width developed between them, only a thin hair crack developed in the bituminous mat. So waterproof was this covering that it became necessary to bore holes through it and the bridge floor in order to drain surface water from low areas. Its condition when inspected in 1938 is shown by figure 9, A.

In contrast to this excellent behavior is that of the adjoining bridge where fabric was not used. As shown in figure 9, B, the surface cracks are much wider.

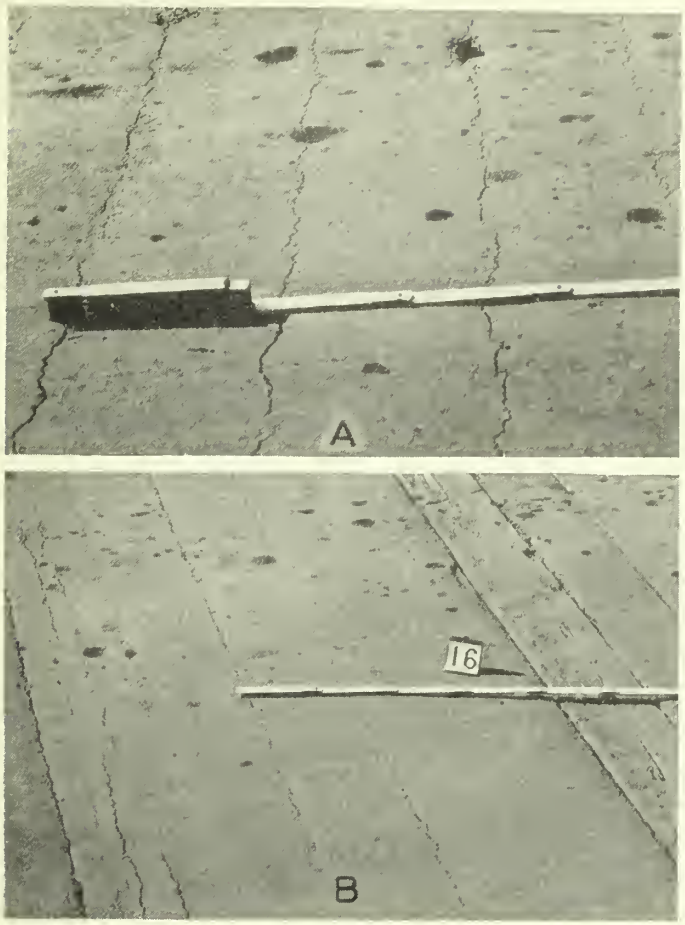

FIGURE 9.—(A) ON A RIGID WOODEN BRIDGE FLOOR, FABRIC DID NOT PREVENT NARROW JOINT CRACKS BUT DID STOP RAVELING OF THE SURFACE. (B) ON A SIMILAR BRIDGE WHERE FABRIC WAS NOT USED THE SURFACE CRACKED AND RAVELED EXTENSIVELY, EXPOSING THE FLOOR PLANKS AND ALLOWING THEM TO ROT.

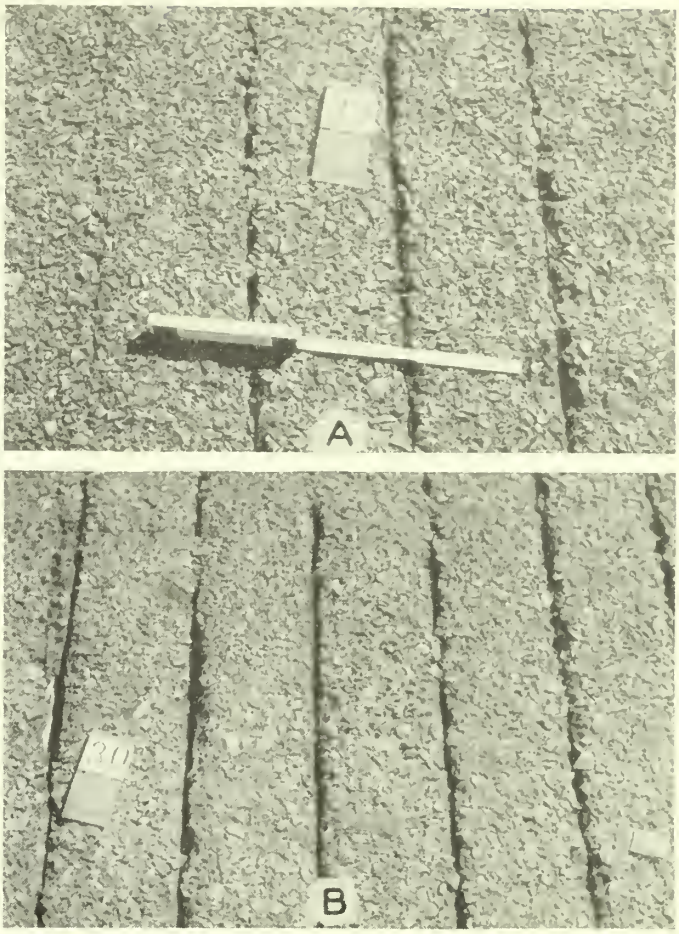

FIGURE 10.—(A) ON A BRIDGE FLOOR WITH LOOSE PLANKS THE FABRIC BROKE QUICKLY. (B) THE CONDITION OF THIS SURFACE ON A SIMILAR BRIDGE IS DUE TO THE EXCELLENT BOND WITH THE PLANKS RATHER THAN THE ABSENCE OF FABRIC.

Raveling occurred in some instances to such an extent that entire planks became uncovered. A number of these had decayed and had been replaced, and at the time the photograph was taken 8 or 10 planks were uncovered and rotten.

No useful purpose appeared to be served by the fabric on bridges that were not rigid or not in good condition when surfaced. The cooperative experimental road in North Carolina included three small wooden bridges which were located a short distance apart and were subjected to identical service conditions. Before the bituminous mat was laid fabric was placed on two of the bridges but not on the third one. The surface condition of two of these bridges 16 months after construction is shown by figure 10, A and B. The mat has adhered well to the planks on all three bridges, but when the planks became loose and moved under traffic the mat broke. On the fabric-covered bridges, the fabric broke sharply whenever the mat cracked because of loose planks. More loose planks were observed on the bridge without fabric than on those containing the fabric. No rotten planks were observed on the latter whereas several were seen on the former. Some of the stringers were showing signs of decay but whether this was the cause of the loose planks or was the result of moisture reaching them after the planks loosened and allowed the mat to crack, is not known. In general, however, where the floor appeared to be rigid the mat laid on the fabric was in better condition than where fabric was not used.

One unsatisfactory condition reported as resulting from the use of fabric on wooden bridge floors was the tendency of the bituminous mat to slip during hot weather. In this instance the fabric was nailed to the planks, which were laid on edge to form the floor, and the bituminous surface was then applied. It was reported that the mat containing the fabric moved under traffic in hot weather and had to be retacked.

PERFORMANCE OF THE EXPERIMENTAL SECTIONS DISCUSSED

The States cooperating in the experimental study provided reports of the construction, periodic condition reports, and annual maintenance costs. The discussion presented herein and the conclusions drawn from this investigation are based upon all of the reports that were submitted including those of inspections by the author which have been given previously.

Some experimental roads were eliminated within 2 years after construction because of reconstruction, abandonment for relocation, or complete failure. Others lost their experimental value as a result of the practice in some States of re-treating all bituminous surfaces annually or biennially. Obviously, little information could be obtained relative to the effect of fabric in a given experimental road reported to be in excellent condition when such condition was due to a

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re-treatment just applied without any explanation of its need, especially when the surface had been previously reported to be in good condition. However, a number of experiments were maintained for the 5-year period agreed to and were re-treated only to the extent necessary to maintain them in the customary condition of serviceability.

The fact that the information obtained varied in completeness does not prevent evaluation of the merits of the fabric, and consequently the conclusions presented herein are believed justified by the rather considerable amount of information that has been obtained.

On the basis of the combined reports obtained from the States and by personal inspection, the information acquired may be classified substantially as follows.

Conclusions were not warranted on seven sections.—On one section construction difficulties and other circumstances made it impossible to evaluate the effect of the fabric.

On one section the fabric was reported to be beneficial in shoulder treatment in contrast to behavior of an adjacent shoulder where fabric was not used. The two areas, however, were of different composition and construction so that comparable conditions did not exist.

On five sections the information submitted was insufficient to permit conclusions.

Fabric appeared to retard but did not prevent surface failures on eight sections.—On two of the sections where the fabric was laid on the wooden floors of bridges it was reported that cracking of the bituminous wearing surface was confined to the plank joints; that the bituminous mat did not ravel; and that the mat, in conjunction with the fabric, continued to prevent surface water from reaching the wooden floor. On adjacent bridges without fabric the mat cracked, raveled, and permitted water to eventually rot the floor plank.

On five road sections it was reported that the fabric, although not preventing the development of cracks, did hold the mat together and delay raveling.

On one section where fabric was placed on the road shoulders, some protection against erosion was reported.

No difference in service behavior of plain and fabric portions was observed on 109 sections.—By far the greater number of sections showed no difference in service behavior regardless of the presence or absence of fabric.

Failures resulting from poor base or drainage conditions on a given section were as prevalent on the fabric portions as on those without fabric.

In fact, the locations of the sections containing fabric could not be identified except by means of the section markers.

It was definitely shown that fabric will not prevent surface failures caused by inferior base material, inadequate thickness, faulty construction, frost action, or cracking of bituminous mats; nor will it prevent moisture reaching the base after the surface cracks. No benefit of fabric as an edge reinforcement was apparent. There was no measurable difference in behavior between the different types of fabric.

Fabric portions were less satisfactory than plain portions on 15 sections.—On two sections fabric was a definite disadvantage from the standpoint of maintenance which consisted of remixing and re-laying the bituminous mats with blade machines.

On 11 sections the surface mats containing fabric did not adhere to the underlying course but slipped laterally under the action of traffic although the fabric had been nailed in place. In three such instances reported the fabric was used on wooden bridge floors and in eight instances the fabric was used in re-treatments applied to existing bituminous surfaces. On the same sections, slipping of the surface mat did not occur when the fabric was not used.

On two sections the fabric portions were simply reported as less satisfactory than the plain portions.

Fabric deteriorated in service on the 12 sections where samples were recovered.—On 12 of the 84 sections personally inspected by the author, samples of the bituminous mats were taken for the recovery and examination of the fabric. Two samples were cut from areas where base and surface failures had occurred and, as might be expected, the fabric had been completely destroyed. All other samples were taken from areas that were in excellent condition and had neither required nor received any maintenance since construction. Without exception the fabric taken from these areas was in very poor condition and in many instances it was practically destroyed. Obviously, the continued good service behavior of the wearing surfaces could not be attributed to the fabric.

CONSTRUCTION AND MAINTENANCE COSTS

The average cost of the fabric delivered and placed was 7.35 cents per square yard or $862 per mile of 20-foot road. This increase in cost would be a substantial percentage of the cost of constructing surface treatments for which the use of fabric was especially advocated and which represented approximately 70 percent of the sections built in this experimental program.

Reports on the maintenance of the experimental roads were submitted through the States by State maintenance engineers and by the local supervisors who were directly responsible for maintaining them. Maintenance practices vary greatly throughout the country,—even in different parts of a given State,—and these variable practices governed the maintenance of the experimental roads to a large extent. A review of the maintenance-cost data submitted indicates chiefly that there was little difference between the maintenance required for the plain sections and that for the sections containing fabric.

CONCLUSIONS

This investigation shows that the benefits expected to result from the use of cotton fabric were not obtained.

No benefit of fabric as an edge reinforcement was apparent. Edge failures are not as prevalent now as formerly and at present they are caused chiefly by unsatisfactory base conditions.

The assumption that the fabric, when impregnated with asphalt or tar, would serve as a waterproof membrane to prevent surface moisture reaching the base did not prove correct as the fabric deteriorated even when surface and base failures did not occur.

Since failures resulting from inadequate base support were as prevalent on sections containing fabric as on those in which fabric was not used, it cannot be said that cotton fabric would compensate for inadequate base support or permit the construction of cheaper bases.

Under certain conditions, as stated herein, cotton fabric was beneficial when used on wooden bridge floors.

The use of cotton fabric in the types of construction employed in these experiments results in a material increase in the cost of construction. It has little or no practical value and consequently the increased cost is not warranted. When used on bridge floors the increased cost may or may not be warranted depending upon the condition of the structure.

TRENDS IN MOTOR-VEHICLE TRAVEL, 1936 TO 1945 [1]

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by G. P. ST. CLAIR, Chief of Division

IN ORDER TO MEET THE DEMAND for information on the subject, classified estimates of motor-vehicle travel in the United States during the past 10 years have been prepared, with the aid of data obtained in cooperative studies by the State highway departments and the Public Roads Administration. The results of these estimates are given in tables 1 to 5.

Table 1 gives the estimates of rural, urban, and total vehicle-miles traveled, in each year from 1936 to 1945, by passenger cars, commercial busses, school and nonrevenue busses, and trucks and combinations. In table 2 are given the numbers of motor vehicles registered annually during the 10-year period, as adjusted for the computation of vehicle-miles. Table 3 gives the travel estimates in terms of annual miles per vehicle. Total motor-fuel consumption classified by type of vehicle is given in table 4, and rates of motor-fuel consumption in miles per gallon are given in table 5.

[1] The data reported in this article are for all rural and urban roads, but exclude military traffic for the years 1942 to 1945. In the article Traffic Trends on Rural Roads in 1945, which appears on page 268 of this issue of PUBLIC ROADS, the travel data reported are for all rural roads and include military traffic; and the data reported as obtained from the summer loadometer surveys are for the 345,000 miles of main rural roads only.

TRENDS IN MOTOR-VEHICLE TRAVEL GREATLY AFFECTED BY WARTIME RESTRICTIONS

The trends in motor-vehicle travel during this 10-year period are illustrated in figures 1, 2, and 3. Figure 1 shows, in line-diagram form, the variation from year to year in the volume of travel — rural, urban, and total — of all motor vehicles taken as a group. Figure 2 shows by similar diagrams the annual variations in rural, urban, and total travel of passenger cars and of trucks and combinations. Annual averages in miles per vehicle are given in figure 3.

The general characteristics of the trends illustrated in these tables and diagrams have been reported by the Public Roads Administration in monthly statements to highway magazines during the past few years. During the years from 1936 to 1941 the trend in motor-vehicle travel was generally upward, culminating in the latter year in a very substantial increase over the levels reached in 1940. Shortly after the outbreak of war the cessation of civilian motor-vehicle production, the rationing of tires, and finally gasoline rationing turned the trend of traffic sharply downward. This

TABLE 1. *Classified estimate of travel by motor vehicles, 1936 to 1945*

Item	Travel in each calendar year in million vehicle-miles									
	1936	1937	1938	1939	1940	1941	1942 [1]	1943 [1]	1944 [1]	1945 [1]
Passenger cars (including taxicabs):										
Rural travel	99,327	106,399	107,786	113,980	120,199	133,891	100,107	71,510	74,321	90,131
Urban travel	109,327	117,068	116,388	121,649	129,060	141,794	118,132	90,278	91,752	109,054
Total	208,654	223,467	224,174	235,629	249,559	275,685	218,239	161,788	166,073	199,185
Commercial busses:										
Rural travel	733	771	771	771	808	878	1,036	1,139	1,327	1,326
Urban travel	1,031	1,083	1,083	1,085	1,136	1,234	1,456	1,601	1,866	1,864
Total	1,764	1,854	1,854	1,856	1,944	2,112	2,492	2,740	3,193	3,190
School and nonrevenue busses:										
Rural travel	542	573	588	627	640	635	573	562	545	577
Urban travel	61	65	66	71	73	73	65	63	61	65
Total	603	638	654	698	713	708	638	625	606	642
All busses:										
Rural travel	1,275	1,344	1,359	1,398	1,448	1,513	1,609	1,701	1,872	1,903
Urban travel	1,092	1,148	1,149	1,156	1,209	1,307	1,521	1,664	1,927	1,929
Total	2,367	2,492	2,508	2,554	2,657	2,820	3,130	3,365	3,799	3,832
All passenger vehicles:										
Rural travel	100,602	107,743	109,145	115,378	121,947	135,404	101,716	73,211	76,193	92,031
Urban travel	110,419	118,216	117,537	122,805	130,269	143,101	119,653	91,942	93,679	110,983
Total	211,021	225,959	226,682	238,183	252,216	278,505	221,369	165,153	169,872	203,017
Trucks and combinations:										
Rural travel	22,076	24,295	25,768	27,771	30,203	34,401	27,145	24,546	24,637	27,149
Urban travel	19,031	19,856	18,727	19,448	19,724	20,490	18,582	17,048	17,071	18,760
Total	41,107	44,151	44,495	47,219	49,927	54,891	45,727	41,594	41,708	45,909
All motor vehicles:										
Rural travel	122,678	132,038	134,913	143,149	152,150	169,805	128,861	97,757	100,830	119,183
Urban travel	129,450	138,072	136,264	142,253	149,993	163,591	138,235	108,990	110,750	129,743
Total	252,128	270,110	271,177	285,402	302,143	333,396	267,096	206,747	211,580	248,926

[1] For years subsequent to 1941, figures do not include travel of vehicles owned by the military services. Data for 1945 are preliminary, subject to revision

TABLE 2.—*Numbers of motor vehicles registered 1936 to 1945 as adjusted for vehicle-mile estimates* [1]

Calendar year	Passenger vehicles					Trucks and combinations	All motor vehicles
	Passenger cars, including taxicabs	Busses			All passenger vehicles		
		Commercial	School and non-revenue	Total			
	1,000 vehicles	*1,000 vehicles*	*1,000 vehicles*	*1,000 vehicles*	*1,000 vehicles*	*1,000 vehicles*	*1,000 vehicles*
1936	24,201	49	75	124	24,325	4,071	28,396
1937	25,490	51	80	131	25,621	4,301	29,922
1938	25,272	51	82	133	25,405	4,285	29,690
1939	26,252	52	87	139	26,391	4,495	30,886
1940	27,488	54	89	143	27,631	4,699	32,330
1941	29,691	59	88	147	29,838	5,112	34,950
1942	27,970	69	80	149	28,119	4,762	32,881
1943	26,005	76	78	154	26,159	4,611	30,770
1944	25,562	80	76	156	25,718	4,642	30,360
1945	25,789	83	80	163	25,952	4,956	30,908

[1] These registration totals differ from those given in the annual Public Roads statistical table MV-1, State Motor-Vehicle Registrations, because of numerous adjustments, chief among which are the following: (1) Inclusion of publicly owned vehicles, which are listed separately in statistical table MV-1; (2) substitution of bus totals as given by the bus industry; and (3) reduction of truck registrations by 2.5 percent to allow for trucks registered in more than one State. For years subsequent to 1941 vehicles owned by the military services are not included.

TABLE 3.—*Estimate of average miles per vehicle traveled by motor vehicles of different types in calendar years 1936 to 1945*

Calendar year	Passenger vehicles					Trucks and combinations	All motor vehicles
	Passenger cars, including taxicabs	Busses			All passenger vehicles		
		Commercial	School and non-revenue	All busses			
	Miles	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*
1936	8,622	36,000	8,000	19,033	8,675	10,098	8,879
1937	8,767	36,000	8,000	18,991	8,819	10,264	9,027
1938	8,871	36,000	8,000	18,822	8,923	10,383	9,134
1939	8,976	36,000	8,000	18,392	9,025	10,504	9,240
1940	9,079	36,000	8,000	18,562	9,128	10,624	9,346
1941	9,285	36,000	7,980	19,134	9,334	10,739	9,539
1942	7,803	36,000	7,989	20,993	7,873	9,602	8,123
1943	6,221	36,000	7,994	21,809	6,313	9,021	6,719
1944	6,497	40,000	7,990	24,401	6,605	8,984	6,969
1945 [1]	7,724	38,500	7,992	23,489	7,823	9,264	8,054

[1] Preliminary estimate, subject to revision.

trend reached its lowest ebb in 1943, when total motor-vehicle travel, amounting to approximately 207 billion vehicle-miles, was only 62 percent of the total in 1941. Although gasoline shortages were somewhat eased in 1944, there was only a slight increase in traffic volume.

With the end of the war in August 1945, the pent-up demand for motor-vehicle travel was in large part released, in spite of continued tire shortages and the scarcity of new passenger cars. Traffic volume surged upward, with the result that 1945 travel, amounting to approximately 249 billion vehicle-miles, was nearly 18 percent above the 1944 level, and about 75 percent of the volume in 1941. This upward trend has continued strongly during recent months.

The major objective of wartime travel restrictions was directed toward reduction in the travel of passenger cars. The effect of this policy can be perceived readily in figure 2. The total volume of passenger-car travel was reduced from 276 billion vehicle-miles in 1941 to 162 billion in 1943, a drop of 41 percent; whereas the travel of trucks and combinations declined only 24 percent, from 55 billion in 1941 to 42 billion in 1943. The volume of travel of commercial busses increased substantially during the war period.

The same trends are shown in figure 3 in which comparisons are given in terms of miles per vehicle. The estimated average annual mileage of passenger cars dropped from 9,285 miles in 1941 to 6,221 in 1943 and recovered to 7,724 in 1945. The drop in average an-

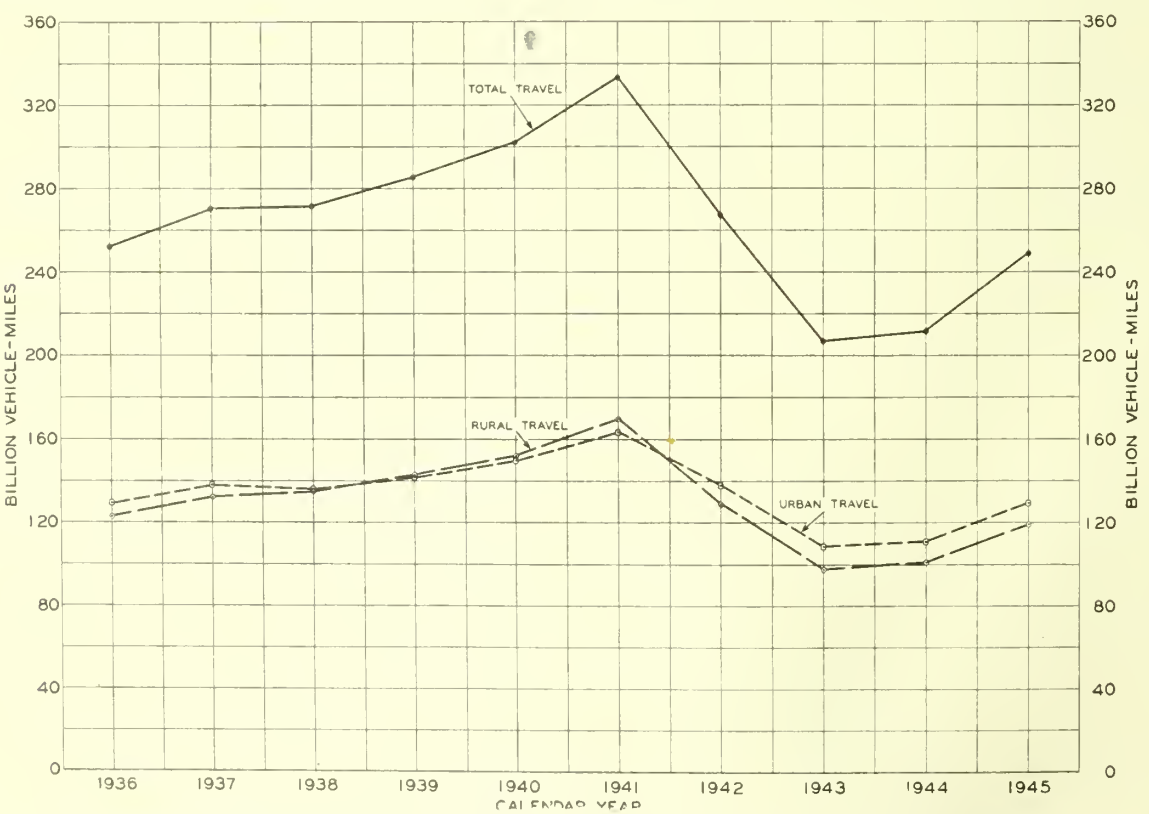

FIGURE 1.—ESTIMATED TRAVEL OF ALL MOTOR VEHICLES IN CALENDAR YEARS 1936 TO 1945.

TABLE 4.—*Estimate of motor-fuel consumption by motor vehicles of different types in calendar years 1936 to 1945*

[In million gallons]

Calendar year	Passenger vehicles					Trucks and combinations	All motor vehicles [1]
	Passenger cars, including taxicabs	Busses			All passenger vehicles		
		Commercial	School and non-revenue	Total			
1936	13,648	320	58	378	14,026	4,003	18,029
1937	14,617	340	61	401	15,018	4,365	19,383
1938	14,663	343	63	406	15,069	4,465	19,534
1939	15,412	347	67	414	15,826	4,807	20,633
1940	16,323	367	69	436	16,759	5,156	21,915
1941	18,031	402	69	471	18,502	5,754	24,256
1942	14,428	484	62	546	14,974	4,889	19,863
1943	10,821	543	60	603	11,424	4,534	15,958
1944	11,108	639	58	697	11,805	4,576	16,381
1945 [2]	13,323	638	62	700	14,023	5,055	19,078

[1] These totals differ by small amounts from the totals of highway use of motor fuel given in the annual Public Roads statistical table G–21, Analysis of Motor-Fuel Usage, chiefly because of an allowance for use in motorcycles.
[2] Preliminary estimate, subject to revision.

TABLE 5.—*Estimate of average miles traveled, per gallon of motor fuel consumed, by motor vehicles of different types in calendar years 1936 to 1945*

[Miles per gallon]

Calendar year	Passenger vehicles				Trucks and combinations	All motor vehicles
	Passenger cars, including taxicabs	Busses		All passenger vehicles		
		Commercial	School and non-revenue	All busses		
1936	15.29	5.50	10.38	6.25	15.04	10.27	13.98
1937	15.29	5.45	10.38	6.20	15.05	10.12	13.94
1938	15.29	5.40	10.38	6.17	15.04	9.97	13.88
1939	15.29	5.35	10.38	6.17	15.05	9.82	13.83
1940	15.29	5.30	10.37	6.10	15.05	9.68	13.79
1941	15.29	5.25	10.36	5.99	15.05	9.54	13.75
1942	15.13	5.15	10.37	5.74	14.78	9.35	13.45
1943	14.95	5.05	10.39	5.58	14.46	9.18	12.96
1944	14.95	5.00	10.39	5.45	14.39	9.12	12.92
1945 [1]	14.95	5.00	10.39	5.48	14.48	9.08	13.05

[1] Preliminary estimate, subject to revision.

nual mileage of trucks and combinations was much more moderate. From a prewar maximum of 10,739 miles in 1941 this average was reduced to 9,021 in 1943 and recovered to 9,264 in 1945.

A comparison of the diagrams given in figure 2 makes it very evident that wartime restrictions had a much greater effect on rural travel than on urban travel. The travel of passenger cars on rural roads in 1943 dropped 47 percent below the 1941 level; whereas their urban travel dropped only 36 percent. Similarly the decline in the rural travel of trucks and combinations was 29 percent, as against a decline of 17 percent in urban travel.

PROCEDURES USED IN ESTIMATING MOTOR-VEHICLE TRAVEL

In planning this analysis it was recognized that rigorous accuracy could not be achieved, and that estimates for the years since 1941 would be particularly vulnerable because the relative amounts of travel by different types of vehicles were profoundly altered during the war by gasoline rationing and other wartime restrictions and shortages. The data available from numerous sources, although gratifyingly consistent in general trends and relations, exhibit inconsistencies in detail which are the result of the variable margins of error surrounding all highway statistics. The values given in tables 1 to 5 represent a compromise solution

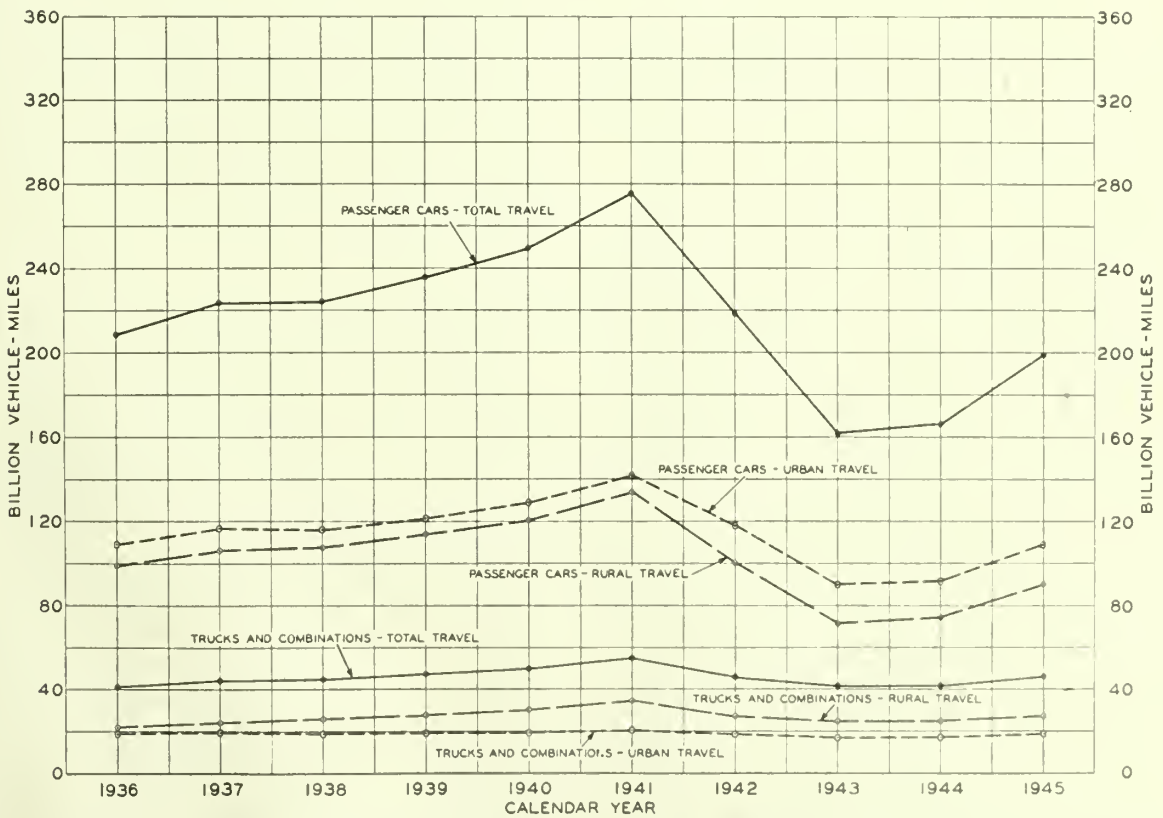

FIGURE 2.—ESTIMATED TRAVEL OF PASSENGER CARS AND OF TRUCKS AND COMBINATIONS IN CALENDAR YEARS 1936 TO 1945.

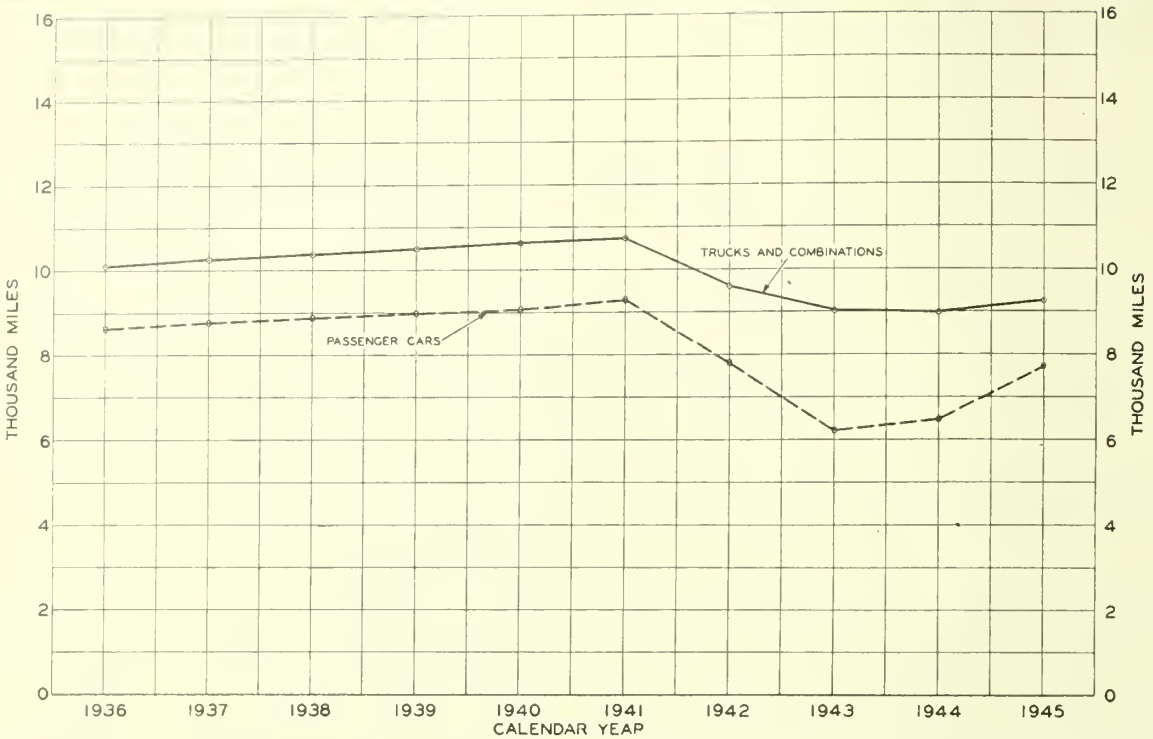

FIGURE 3.—ESTIMATED AVERAGE MILES PER VEHICLE TRAVELED BY PASSENGER CARS AND BY TRUCKS AND COMBINATIONS IN CALENDAR YEARS 1936 TO 1945.

in which conflicts in the basic data were resolved, sometimes by averaging and sometimes by the exercise of judgment. Some of the principal features of the analysis are described in the following paragraphs.

Basic Sources of Data.—Data regarding the volumes of travel by motor vehicles of different types are obtainable from four major types of study which the Public Roads Administration and the planning surveys of the State highway departments have conducted during recent years. These are the traffic surveys, the motor-vehicle-allocation surveys, the road-use surveys, and the truck and bus inventory.

The comprehensive basic surveys of rural-road traffic, covering a single year, were conducted in most States during the period from 1935 to 1939. For any individual State, estimates for years since that of the comprehensive survey are made on a comparative basis by means of the continuous record of automatic recorder stations, supplemented by whatever classification counts the State has made, including the annual summer counts in which data on vehicle loads are also obtained. The results of these continuous studies, which have been published by the Public Roads Administration in Wartime Changes in the Volume and Composition of Traffic on Rural Roads in the United States (1942, 1943, 1944), yield among other things Nation-wide estimates of the total volume of rural-road traffic classified as between "Trucks and Combinations" and "Other", the latter in effect being passenger vehicles.

The motor-vehicle-allocation studies, which also were 1-year studies, conducted in most States during the period from 1935 to 1939, were originally designed as an adjunct to the planning-survey fiscal studies for the purpose of determining the incidence of motor-vehicle taxation with respect to vehicle type and situs of ownership (rural, urban, and size of urban place). In order to evaluate gasoline-tax payments the respondents, representing a sample of the motor-vehicle population, were asked to report actual or estimated miles driven and average miles per gallon during the previous year. A byproduct of the sample expansion was an estimate of the total miles driven by vehicles registered in the State during the year under study.

The road-use surveys also were designed as adjuncts of the fiscal studies with the objective of determining the incidence of benefit of highway expenditures with respect to vehicle type and situs of ownership. These were interview studies in which the respondents, representing a sample of the motor-vehicle population, were asked to report actual or estimated miles traveled during the previous year, and to account for this driving by describing customary trips, such as home-to-work driving; unusual trips, such as vacation driving; and incidental pleasure and business driving. With the aid of highway maps carried by the interviewers, the trips thus described were subdivided into travel on primary rural roads, secondary and local rural roads, city streets, and "out-of-State." The result was an estimate of the respondent's travel during the year subdivided by class of road or street on which the travel was performed. Other classifications obtained, such as trip length and purpose of travel, are not germane to this discussion. Expansion of the sample led to an approximate evaluation of total travel by vehicles registered in the State during the year under study, classified by type of vehicle and class of road.

From the above description it is evident that the traffic studies produce estimates of rural travel classified by vehicle type, the motor-vehicle-allocation studies produce estimates of total travel classified by vehicle type, and the road-use studies produce estimates of total travel subject to both rural-urban and vehicle-type classifications. As might be expected, the results of these three independent studies, all of them founded on sampling techniques, were not entirely consistent in

any individual State or in Nation-wide totals developed from the State figures.

In connection with the preparation of table 1, a calculation was made to determine the relative magnitudes of the changes which would be made, (1) in the volume and distribution of rural travel given by the traffic studies and (2) in the volume and distribution of total travel given by the motor-vehicle-allocation studies, if each were forced to meet "halfway" the rural-urban distribution given by the road-use studies. This comparison was made for the year 1936 which was the year covered by the greatest number of planning survey studies. The percentage changes indicated by this calculation were as follows:

	Traffic data (rural)	*Motor-vehicle-allocation data (total)*
Passenger vehicles	+1.31	−3.03
Trucks and combinations	+5.15	+3.29
All motor vehicles	+2.00	−2.00

It was evident that, so far as Nation-wide totals are concerned, a mutual adjustment of the results of all three studies could be made without doing great violence to any of them. In the calculations underlying table 1 it was decided to adopt the following procedure: (1) To accept unchanged the data on rural-road travel given by the traffic studies; (2) to evaluate the distribution of total travel on the basis of the motor-vehicle-allocation studies (urban travel resulting as the difference between the two); and (3) to use road-use data only in auxiliary computations and as a check on the consistency of the results.

One factor leading to this decision was that the elaborate process of reconciliation could thereby be eliminated. Furthermore, a reconciliation which would be acceptable to all concerned with the three studies would have to be made on a State-by-State basis, necessitating a tremendous volume of work. The most compelling reason, however, for accepting the traffic data unchanged is that the automatic recorder data and periodic classification counts afford a means of determining the trends in recent years, whereas there is no corresponding series of periodic data whereby the rural-urban distribution given by the road-use surveys could be corrected for changes since the time when they were made.

The data given by the truck and bus inventory, which are characteristic of the year 1940-41, were used to check the consistency of the calculations, and also for certain auxiliary computations. The annual published data on motor-vehicle registration and gasoline consumption were used as a means of carrying forward the estimates of total travel from the basic year, 1936, to the final year, 1945.

Motor-Vehicle Registrations.—In preparation for the evaluation of vehicle-miles a number of adjustments were made in the motor-vehicle registration statistics published in the annual Public Roads table MV–1. Certain inconsistencies in the figures reported by individual States, such as the failure to segregate trailers from trucks or the inclusion of light trucks with passenger cars, were corrected by means of approximations based on data from planning survey studies or other sources. In the case of busses, the annual motor-vehicle statistics do not provide a complete segregation of commercial busses from school and other nonrevenue busses. For this reason the bus totals given in the Public Roads tables were replaced by the totals reported by the bus industry modified only by adding the small number of busses reported as owned by the Federal Government. In the case of trucks and combinations it was thought that a reduction in the reported totals should be made in order to allow for vehicles of this type registered in more than one State. A reduction factor of 2.5 percent, based on data obtained in the truck and bus inventory, was adopted. The adjusted registration totals as given in table 2 include publicly owned vehicles—Federal, State, and local. For years subsequent to 1941, vehicles owned by the military services are not included.

Busses and Federal Vehicles.—In order to simplify the calculations, estimates for busses and for motor vehicles owned by the Federal Government were made separately from the prorating computations (discussed in a later section) by means of which year-to-year estimates of passenger-car and truck travel were made.

In order to estimate the vehicle-miles traveled and motor fuel consumed by busses, it was necessary to determine reasonable average values of annual mileage and miles per gallon. Comparisons were made of data regarding busses given by the motor-vehicle-allocation studies, the truck and bus inventory, and the statistics of the bus industry. No consistent trend in the annual mileage of busses was observed in any of the data available. The values of 36,000 miles per year for commercial busses and 8,000 miles per year for school and nonrevenue busses were, however, found to be reasonably consistent with all the sources, and these values were used for the years 1936 through 1943. Bus industry figures indicated greater annual mileages for commercial busses in 1944 and 1945. The 1944 estimate was set at 40,000 miles and the preliminary estimate for 1945 at 38,500. The estimate of 8,000 miles for school and nonrevenue busses was carried unchanged through 1945.

For 22 States reporting data for commercial busses separately in the motor-vehicle-allocation studies the weighted average per gallon for in-State travel was 5.9 miles. The magazine Bus Transportation publishes each year a report of the total revenue-miles and the total gasoline consumption of common-carrier regular-route operators. Miles per gallon computed from these figures for a number of years were found to vary from 4.4 to 4.6, with some indication of a downward trend. By making an approximate allowance for nonrevenue mileage and taking account of the mileage and gasoline consumption of other commercial busses, estimates were made which varied from 5.38 miles per gallon in 1937 to 5.04 in 1943. After comparison of these figures with the motor-vehicle-allocation data, it was decided to use for commercial busses the series given in table 5 which varies from 5.50 miles per gallon in 1936 to 5.00 miles in 1944 and 1945. This downward trend finds confirmation in bus-industry figures, which indicate a very great increase in the numbers of revenue passengers per bus during this 10-year period.

In the case of school and nonrevenue busses the average of 10.4 miles per gallon given by the motor-vehicle-allocation studies was used for all years.

The computations for Federally owned vehicles, which contribute a negligible part of the total travel, were based largely on the findings regarding annual mileage and miles per gallon resulting from the reports made by all Federal agencies in recent years on the operation of vehicles owned by them. The inclusion in the assembled totals of data computed separately

for Federal vehicles has the effect of altering slightly some of the basic averages of annual mileage and miles per gallon used in the calculations for vehicles other than Federal.

Miles Per Gallon for Passenger Cars and Trucks.—The weighted average miles per gallon given by the motor-vehicle-allocation studies were 15.575 for passenger cars and 10.511 for trucks and combinations. These values have been regarded by numerous critics as somewhat too high, largely on the ground that any bias in an individual's estimate of miles per gallon is likely to be on the high side. It is also true that data for heavy trucks, including combinations, were rather meager and somewhat erratic. In making compromises to reconcile data from various sources the following modified values were determined for the year 1936:

	Miles per gallon	*Percentage change*
Passenger cars	15.288	−1.88
Trucks and combinations	10.256	−2.49

For passenger cars this value was held constant through 1941, but the values for the years thereafter were reduced slightly to allow for the higher percentage of urban driving and reduced efficiency of operation resulting from gasoline rationing and other wartime conditions. The values taken for 1942 and for subsequent years were 15.125 and 14.950, respectively.

For trucks and combinations a correction was introduced to allow for the very substantial increases in average loads observed in loadometer studies made in recent years. From data in the Wartime Changes bulletins, estimates were made of the percentage increases in average operating gross weight between 1936 and 1943. Since the loadometer studies were made on main rural highways, it was estimated that the percentage changes applicable to all roads and streets were two-thirds as great as those indicated by the loadometer studies. The corresponding changes in miles per gallon were obtained by reference to an approximate relation between average operating gross weight and miles per gallon developed in a previous study. This relation, based on a correlation between motor-vehicle-allocation and loadometer data in a number of States, is expressed by the equation:

$$g = 0.001137\,W^{0.50177}$$

Where g = gallons per mile, and
W = average operating gross weight in pounds

Although this equation is based on the analysis of a very limited amount of data, it was considered sufficiently accurate for use in estimating the changes in average miles per gallon of trucks and combinations since the base year, 1936. The final values obtained by this means were as follows:

1936	10.256	1941	9.494
1937	10.100	1942	9.334
1938	9.946	1943	9.165
1939	9.800	1944	9.107
1940	9.654	1945	9.075

The slight differences between the values given above and those listed for trucks and combinations in table 5 resulted from the inclusion of data for Federal vehicles in the final calculation of averages.

Vehicle-mile Computations, Passenger Cars and Trucks.—The prorating procedure by means of which vehicle-miles were evaluated year by year from 1936 through 1941 for the two major vehicle types, passenger cars and trucks and combinations, was as follows:

TABLE 6.—*Comparison of rural travel in the years 1941 to 1943*

Type of vehicle	Rural travel			Comparison by years		
	1941	1942	1943	1942/41	1943/41	1943/42
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Percent*	*Percent*	*Percent*
Passenger vehicles	135,404	101,716	73,211	75.12	54.07	71.78
Trucks and combinations	34,401	27,145	24,546	78.91	71.35	90.43
Total	169,805	128,861	97,757	75.89	57.57	75.86

(1) The net gasoline consumption by passenger cars and trucks as a group was determined for each year by deducting the amounts consumed by Federal vehicles and busses, computed separately, and a small allowance for motorcycles, from the published annual totals of highway use (Public Roads tables G–21 and G–201).

(2) For the year 1936 trial values of gasoline consumption by passenger cars and trucks, respectively, were computed by applying to the registration totals the weighted average values of annual mileage given by the motor-vehicle-allocation studies (8,665.8 for cars and 10,131.0 for trucks) and the adjusted average values of miles per gallon (15.288 for cars and 10.256 for trucks); and these values were added to a trial total.

(3) The trial values of gasoline consumption for passenger cars and trucks, respectively, were adjusted by the factor necessary to make their sum equal the "true" value of gasoline consumption by passenger cars and trucks as determined in (1).

(4) Vehicle-miles were evaluated by multiplying the adjusted values of gasoline consumption by the given values of miles per gallon; and adjusted values of average mileage were determined.

(5) For each subsequent year through 1941 the process was repeated, the trial values of average mileage being the final values determined for the previous year.

The assumption underlying this procedure is that changes in total gasoline consumption not accounted for by changes in the number of registered vehicles (together with the estimated changes in the miles per gallon of trucks) are attributable to proportional increases in the average annual mileages of passenger cars and trucks.

Procedure Used for 1942 and Subsequent Years.—It was necessary to devise a different procedure for these years because of the known differences in the relative effects on passenger cars and trucks of gasoline rationing and other wartime restrictions and shortages. These differential effects are illustrated by the rural traffic data shown in table 6.

Among those who follow the trends of traffic movement it is a matter of general agreement, supported by a considerable volume of data, that the wartime effects on urban travel were less severe than on rural travel. Urban holders of A gasoline rations did very little rural driving. Urban and suburban holders of home-to-work rations undoubtedly reduced their rural driving in great measure. With respect to trucks the relative changes were not so clearly indicated, but the evidence of urban toll facilities and a limited amount of urban street data tend to confirm the general impression that urban truck travel did not suffer as great a reduction as did rural truck travel.

Since the data on urban traffic trends were not sufficient either in volume or in geographical distribution to establish reliable Nation-wide averages, it was

TABLE 7.—*Effects on calculation of urban travel of assumptions as to the relation of urban travel to rural travel*

Conditions	Rural travel	Urban travel	Total travel
1941 travel.....million vehicle-miles	169,805	163,591	333,396
No change in urban travel (1):			
1942 travel.....million vehicle-miles	128,861	163,591	292,452
Relation, 1942/1941.....percent	75.89	100.00	87.72
Urban change proportional to rural change (2):			
1942 travel.....million vehicle-miles	128,861	124,149	253,010
Relation, 1942/1941.....percent	75.89	75.89	75.89
Urban change one-half of percentage of rural change (3):			
1941 travel.....million vehicle-miles	128,861	143,862	272,723
Relation, 1942/1941.....percent	75.89	87.91	81.80
Urban change two-thirds of percentage of rural change (4):			
1942 travel.....million vehicle-miles	128,861	137,302	266,163
Relation, 1942/1941.....percent	75.89	83.93	79.83
Deviations of estimates (3) and (4) in percentage of lower value		4.8	2.5

decided to use the rural traffic data as a guide in making the urban estimates. The procedure, which was applied separately to passenger cars and to trucks and combinations, was that of estimating the year-to-year change in urban travel as a percentage of the corresponding change in rural travel. The possibilities of error in this somewhat arbitrary procedure were held within rather narrow limits by the further requirement that the final estimate of total vehicle-miles for each year should be consistent, in terms of motor-fuel consumption accounted for, with the annual published figures on highway use of motor fuel.

In considering estimates for 1942 one might make various assumptions on the decline in urban travel in percentage of its 1941 value. The outer limits of such an estimate might be taken as (1) no reduction at all in urban travel, and (2) a percentage reduction equal to that of rural travel. More reasonable assumptions of a percentage decline in urban travel would be that it equalled (3) one-half, or (4) two-thirds, of the decline in rural travel. The results of calculations according to such assumptions are shown in table 7.

With respect to total travel in 1942, the difference between the results of the two extreme assumptions (1) and (2) is 39,442 or 15.6 percent of the smaller number. The urban estimates differ by 31.8 percent.

It is apparent that, within the range of the more reasonable assumptions (3) and (4) regarding the relative changes in rural and urban travel, very small deviations in the estimates are to be expected. The margin of error was still further narrowed by evaluating motor-fuel consumption from the assumed vehicle-miles and then applying the prorating process so as to equate to the reported highway use of motor fuel as given in Public Roads statistical table G–21.

In making the estimates for 1942 the following assumptions were used: Urban travel of passenger vehicles experienced two-thirds the percentage reduction from 1941 experienced by their rural travel; and urban travel of trucks and combinations experienced one-half the percentage reduction from 1941 experienced by their rural travel.

This device of predicating the estimates of urban travel upon the estimates of rural travel derived from the traffic studies was carried through for the years 1943, 1944, and 1945, the estimates of urban travel for a given year being based on the changes in rural travel from that of the preceding year.

Although the procedure described above, controlled as it is by the reported total volume of motor-fuel consumption, has produced very reasonable results for a series of 4 years, it cannot be recommended as a means for making continuing annual estimates over an extended period. A more firm basis should be established for estimating the total volume of urban travel.

Travel of Military Vehicles. In the estimates for 1941 and prior years vehicles owned by the military services were included with other publicly owned vehicles, and their travel is therefore included in the estimates given in table 1. For 1942 and subsequent years no information is publicly available regarding either the numbers of military vehicles or the motor fuel consumed by them, and the totals and averages given for these years in tables 1 to 5 are applicable to civilian motor vehicles only.

Since the rural traffic studies on which the estimates of rural-road travel are based involve the counting of all vehicles whether civilian or military, it was necessary in arriving at final estimates of the total travel by civilian vehicles to adjust for the relative numbers of military vehicles observed in the traffic counts.

In the annual summer loadometer and traffic studies made on main rural roads in recent years the following Nation-wide percentages of military vehicles to total vehicles were observed:

1942	1.08706	1944	1.14327
1943	1.92865	1945	0.81758

The percentages given above were applied to the estimated values of all vehicle-miles on main rural roads,[2] and the resulting estimates of the travel of military vehicles on main rural roads were deducted from the estimated total vehicle-miles on all rural roads. Because of the fact that the data for civilian trucks and combinations are given separately in the rural traffic estimates, this deduction was applicable to the totals for vehicles other than civilian trucks and combinations, the estimates for the latter remaining unchanged. The amounts of these deductions, representing the estimated travel of military vehicles on main rural roads, were:

	Million vehicle-miles		*Million vehicle-miles*
1942	1,015	1944	1,049
1943	1,370	1945	701

It might have been preferable to estimate travel of military vehicles on secondary and local roads and on city streets so as to include the travel of all vehicles, civilian and military, in table 1; but the data on which to base such estimates were entirely lacking.

Revisions of Vehicle-mile Estimates.—For the years 1936 to 1940 the estimates given in tables 1, 2, and 3 are identical with the estimates previously issued in statistical table VM–1 for 1945. The data for the years 1941 to 1944 have been revised, the most notable change being the adjustment with respect to the travel of military vehicles in the years 1942 to 1945 described in the preceding section. The previously published figures for 1944 were preliminary, and are replaced by revised estimates. Other minor revisions were made in the interest of increased accuracy.

The estimates for 1945 are indicated as preliminary, subject to revision at the time preliminary estimates for 1946 are made. It is believed, however, that no material changes will result from such a revision since most of the data on which the 1945 estimates were based were in final form.

[2] See table 11, page 15, in Wartime Changes in the Volume and Composition of Traffic on Rural Roads in the United States in the year 1944

TRAFFIC TRENDS ON RURAL ROADS IN 1945 [1]

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by THOMAS B. DIMMICK, Highway Economist

THE TRAFFIC VOLUME TRENDS reported in this analysis are derived principally from the records received from approximately 625 fixed automatic traffic-recorder stations operated continuously on rural roads in 46 States. Trends concerning the volume and nature of truck traffic and loadings were obtained from the information collected in the summer surveys described in another part of this article. Supplemental counts made by many States yielded valuable information concerning total traffic and the portion of the total that consists of single-unit trucks and of combinations. In the analysis of the data consideration has been given to all available information, but the most reliable of this—that derived from the traffic sample with the most complete coverage—is given preference in this report. Vehicle-mile figures for State systems prepared by the States for current years have been used when available in preference to estimates made by applying trend data to estimates of previous years.

[1] The travel data reported in this article are for all rural roads and include military traffic. The data reported as obtained from the summer loadometer surveys, however, are for the 345,000 miles of main rural roads only. In the article Trends in Motor-Vehicle Travel, 1936 to 1945, which appears on page 261 of this issue of PUBLIC ROADS, the data reported are for all rural and urban roads, but exclude military traffic for the years 1942 to 1945.

TRAVEL TRENDS ON ALL RURAL ROADS

The seasonal variation of all travel on all rural roads for the years 1940 to 1945, inclusive, is shown in figure 1. Traffic in 1943 and 1944 was reduced to about 60 percent of the prewar level, but in 1945 it gradually increased with the approaching end of hostilities until in September with the termination of gasoline rationing it reached a level not much below that of 1940.

Figure 2 shows the relation of traffic on all rural roads in 1944 and in 1945 to that in corresponding months in 1941. As might be expected, a large increase of traffic

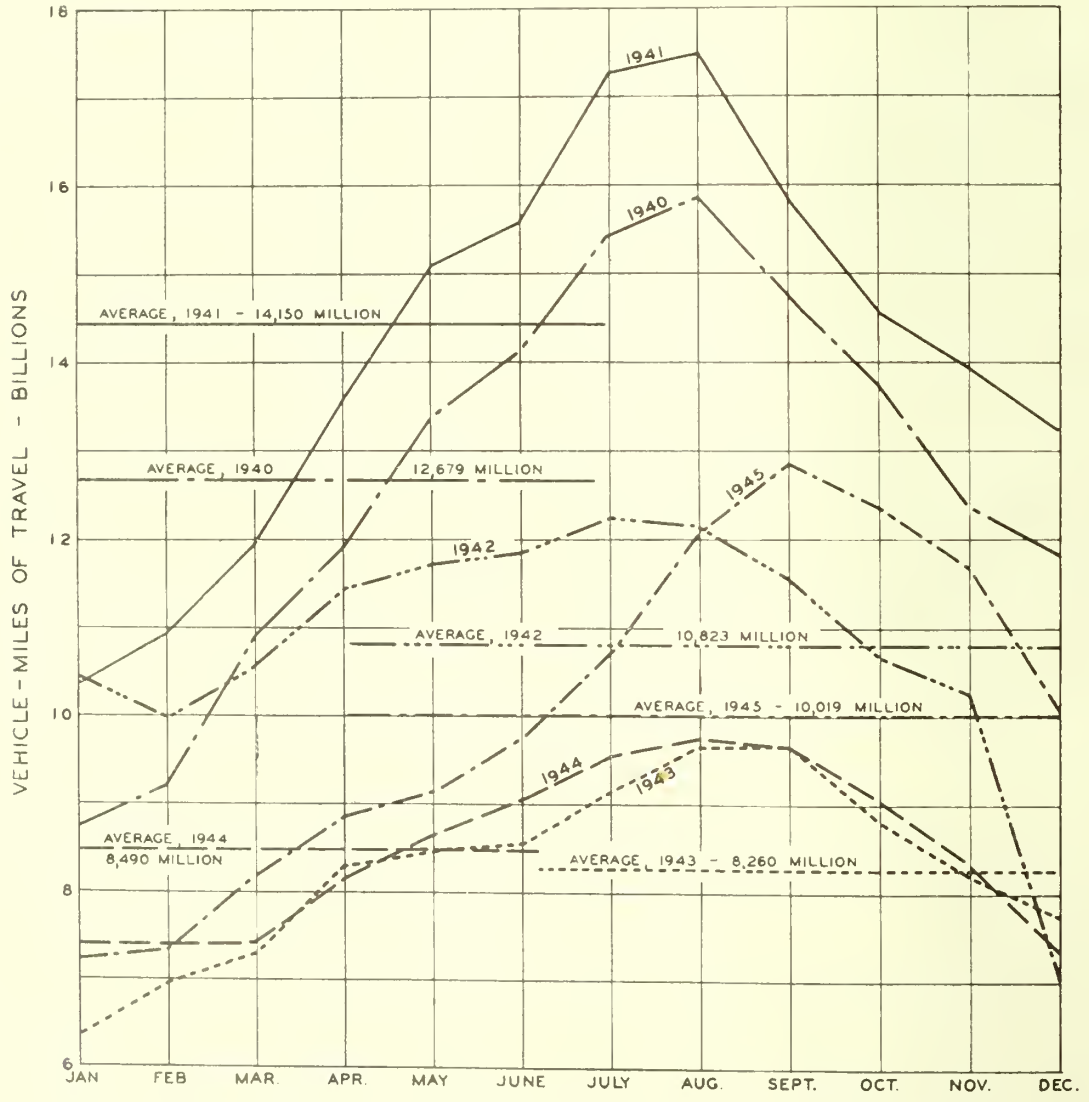

FIGURE 1.—VEHICLE-MILES OF TRAVEL ON ALL RURAL ROADS FROM 1940 TO 1945 BY MONTHS.

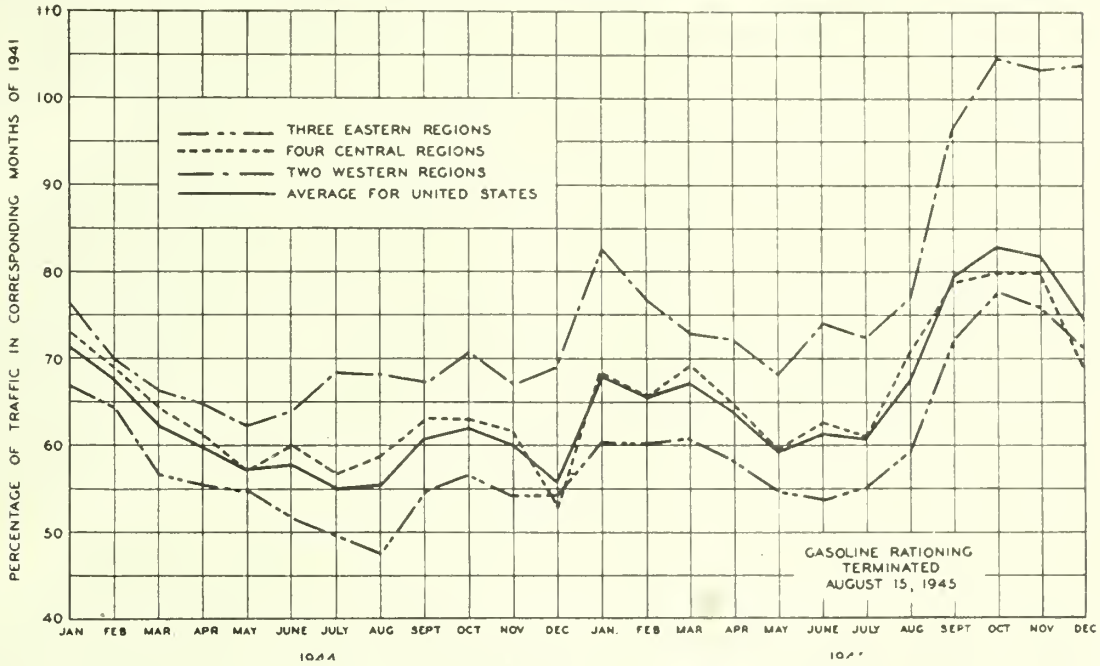

FIGURE 2.—RELATION OF RURAL TRAFFIC IN 1944 AND 1945 TO THAT IN THE CORRESPONDING MONTHS OF 1941 EXPRESSED AS PERCENTAGES.

occurred in all sections of the country as soon as gasoline rationing was terminated. Particularly noteworthy is the increase of traffic in the western regions, where total traffic increased so rapidly that during the last 2 months of 1945 it exceeded the previous all-time levels established in 1941.

Traffic information is analyzed in this report for the United States as a whole, for each census region, and generally for three groups of regions that roughly represent the eastern seaboard, the Central States, and the Western States. The census regions with the States included in each are shown in figure 3. The data available are not deemed sufficient to justify presentation of separate statistics for each State.

The ratio of traffic volume in 1945 to that in corresponding months of 1944 is shown in table 1. The general traffic level in 1945 increased from a point 5 percent below the level of January 1944 to 33 percent above that of December 1944.

In table 2 the 1945 traffic is compared with that in the corresponding months of 1941. The approach to 1941 levels during the latter part of 1945 in all sections of the country, and exceeding this level in the Pacific region, is evident.

Table 3 shows by census regions the percentage of the year's traffic in each month of the year, for 1945 and for 1941. Of interest is the noticeable shifting of traffic from the Eastern States to the Pacific Coast area that occurred during the period. Only 9.70 percent of the total traffic in the United States occurred in the Pacific region in 1941 whereas in 1945 the percentage for this region increased to 12.13. The percentage for the three eastern regions, on the other hand, decreased from 33.83 percent in 1941 to 30.83 percent in 1945.

FIGURE 3.—REGIONS OF THE UNITED STATES BASED ON GROUPINGS OF THE STATES BY THE U. S. BUREAU OF THE CENSUS.

SUMMER SURVEY MADE ON MAIN RURAL ROADS

During the summer of 1945 a survey was conducted by the highway departments of 37 States in cooperation with the Public Roads Administration to determine the trends in the volume and composition of traffic and in the weights of trucks and truck combinations

TABLE 1.—*Ratio of traffic volume in 1945 to that in corresponding months of 1944 as determined from automatic traffic-recorder data*

Region	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Eastern regions:												
New England	0.86	0.88	1.01	1.03	0.97	1.10	1.14	1.31	1.41	1.38	1.36	1.23
Middle Atlantic	.84	.93	1.10	1.05	.99	1.01	1.10	1.28	1.34	1.40	1.41	1.32
South Atlantic	.97	.96	1.07	1.07	1.01	1.04	1.09	1.19	1.26	1.35	1.39	1.32
Average	.90	.93	1.07	1.05	1.00	1.04	1.11	1.24	1.32	1.37	1.40	1.31
Midwest regions:												
East North Central	.87	.93	1.09	1.05	1.01	1.02	1.06	1.22	1.25	1.28	1.29	1.34
East South Central	.93	.94	1.03	1.02	1.06	1.05	1.14	1.26	1.26	1.27	1.34	1.34
West North Central	.95	.98	1.14	1.09	1.05	1.08	1.04	1.17	1.29	1.28	1.25	1.19
West South Central	1.03	.99	1.03	1.04	1.05	1.04	1.11	1.16	1.21	1.23	1.30	1.30
Average	.94	.96	1.08	1.05	1.04	1.04	1.07	1.20	1.25	1.27	1.29	1.29
Western regions:												
Mountain	1.08	1.08	1.09	1.09	1.12	1.13	1.16	1.24	1.32	1.38	1.37	1.33
Pacific	1.09	1.11	1.11	1.13	1.08	1.17	1.17	1.23	1.49	1.53	1.63	1.58
Average	1.09	1.10	1.10	1.12	1.10	1.16	1.17	1.23	1.43	1.47	1.54	1.50
United States average	.95	.97	1.08	1.06	1.04	1.06	1.10	1.22	1.30	1.34	1.36	1.33

TABLE 2.—*Ratio of traffic volume in 1945 to that in corresponding months of 1941 as determined from automatic traffic-recorder data*

Region	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Eastern regions:												
New England	0.61	0.56	0.62	0.54	0.47	0.46	0.45	0.49	0.69	0.68	0.63	0.61
Middle Atlantic	.57	.59	.54	.57	.51	.50	.55	.60	.70	.76	.73	.67
South Atlantic	.63	.62	.66	.62	.62	.61	.60	.64	.76	.84	.83	.78
Average	.60	.60	.61	.58	.55	.54	.55	.59	.72	.78	.76	.71
Midwest regions:												
East North Central	.61	.62	.66	.57	.52	.55	.54	.67	.76	.73	.73	.63
East South Central	.75	.73	.78	.74	.70	.73	.70	.85	.84	.89	.94	.71
West North Central	.74	.65	.68	.63	.59	.62	.59	.64	.78	.81	.74	.64
West South Central	.71	.68	.72	.75	.72	.76	.74	.79	.83	.86	.91	.79
Average	.69	.66	.69	.65	.60	.63	.61	.71	.79	.80	.80	.68
Western regions:												
Mountain	.81	.76	.70	.72	.68	.67	.66	.74	.89	.96	.89	.90
Pacific	.84	.77	.75	.72	.69	.78	.76	.78	1.00	1.09	1.12	1.11
Average	.83	.77	.73	.72	.68	.74	.73	.77	.96	1.05	1.03	1.04
United States average	.68	.66	.67	.64	.59	.61	.61	.68	.79	.83	.82	.74

on main rural roads. This survey followed the general plan of similar surveys conducted in 1942, 1943, and 1944. Weighing stations were operated during July, August, or September in all States except Florida, Indiana, Massachusetts, Missouri, Oregon, Ohio, Rhode Island, Utah, Vermont, South Carolina, and Virginia.

During the survey 441 weight stations were operated, all of which had been used in the 1944 survey except 10 in Iowa which had been used in the survey conducted in 1943. The work was so scheduled as to insure maximum comparability with data obtained in the previous years.

Each station was operated for 8 hours on a weekday, either from 6 a. m. to 2 p. m. or from 2 p. m. to 10 p. m. All traffic passing the stations during the period of operation was counted and classified. Military vehicles were separated from civilian traffic and classified into six types. Civilian vehicles were classified as local passenger cars, foreign or out-of-State passenger cars, light single-unit trucks, medium single-unit trucks, heavy single-unit trucks, tractor-truck and semitrailer combinations, truck and trailer combinations, and busses.

During the period of the survey 3,979 military vehicles and 482,702 civilian vehicles were counted. The civilian traffic included 102,598 trucks or 21.3 percent of the total. The survey period, number of stations operated, number of vehicles counted, and number of trucks weighed in each State are shown in table 4.

At stations where traffic volume permitted all trucks and truck combinations were stopped and weighed, but where the volume was so large as to make this procedure impracticable a representative sample was obtained by selecting trucks at random from those which passed. Military vehicles, passenger cars, and

TABLE 3.—*Percentage of total monthly traffic in each United States census region in 1945 and 1941 with ratio between the percentage in 1945 and the percentage in 1941*

Period	Eastern regions				Midwest regions					Western regions			United States total
	New England	Middle Atlantic	South Atlantic	Total	East North Central	East South Central	West North Central	West South Central	Total	Mountain	Pacific	Total	
January:													
1941	4.54	12.82	16.10	33.46	19.18	6.63	11.73	13.83	51.37	5.04	10.13	15.17	100
1945	4.05	10.75	14.94	29.74	17.19	7.36	12.82	14.39	51.76	5.99	12.51	18.50	100
1945:1941	.89	.84	.93	.89	.90	1.11	1.09	1.04	1.01	1.19	1.23	1.22	
February:													
1941	4.99	12.27	15.54	32.80	19.27	6.51	12.66	13.20	51.64	5.11	10.45	15.56	100
1945	4.24	11.09	14.73	30.05	18.27	7.24	12.52	13.75	51.78	5.88	12.28	18.17	100
1945:1941	.85	.90	.95	.92	.95	1.11	.99	1.04	1.00	1.15	1.18	1.17	
March:													
1941	4.95	12.76	14.83	32.55	20.06	6.33	12.50	12.91	51.79	5.59	10.07	15.66	100
1945	4.60	10.25	14.63	29.47	19.76	7.40	12.56	13.80	53.52	5.82	11.18	17.01	100
1945:1941	.93	.80	.99	.91	.99	1.17	1.00	1.07	1.03	1.04	1.11	1.09	
April:													
1941	5.53	14.25	14.50	34.28	20.95	6.10	12.36	11.53	50.95	5.06	9.72	14.77	100
1945	4.71	12.64	14.08	31.43	18.80	7.07	12.28	13.64	51.79	5.72	11.06	16.78	100
1945:1941	.85	.89	.97	.92	.90	1.16	.99	1.18	1.02	1.13	1.14	1.14	
May:													
1941	5.86	15.01	13.10	33.97	21.65	5.98	12.66	10.61	50.90	5.19	9.94	15.13	100
1945	4.68	13.03	13.66	31.36	18.84	7.03	12.50	12.85	51.22	5.92	11.49	17.42	100
1945:1941	.80	.87	1.04	.92	.87	1.18	.99	1.21	1.01	1.14	1.16	1.15	
June:													
1941	5.93	14.87	13.39	34.19	21.29	6.00	12.62	11.03	50.94	5.40	9.47	14.87	100
1945	4.42	12.10	13.41	29.93	19.09	7.12	12.76	13.15	52.12	5.91	12.04	17.95	100
1945:1941	.75	.81	1.00	.88	.90	1.19	1.01	1.19	1.02	1.09	1.27	1.21	
July:													
1941	6.42	14.87	13.47	34.76	21.80	6.01	12.31	10.12	50.25	5.47	9.52	14.99	100
1945	4.74	13.48	13.31	31.53	19.24	6.94	11.94	12.42	50.54	5.94	11.99	17.93	100
1945:1941	.74	.91	.99	.91	.88	1.15	.97	1.23	1.01	1.09	1.26	1.20	
August:													
1941	6.56	15.55	13.21	35.31	21.40	5.81	12.63	10.11	49.96	5.30	9.43	14.73	100
1945	4.77	13.75	12.44	30.96	21.26	7.28	12.01	11.76	52.31	5.81	10.92	16.73	100
1945:1941	.73	.88	.94	.88	.99	1.25	.95	1.16	1.05	1.10	1.16	1.14	
September:													
1941	6.04	14.02	13.06	34.02	21.35	6.21	12.54	11.04	51.14	5.12	9.72	14.84	100
1945	5.24	13.25	12.52	31.02	20.35	6.61	12.42	11.55	50.92	5.75	12.31	18.06	100
1945:1941	.87	.95	.96	.91	.95	1.06	.99	1.05	1.00	1.12	1.27	1.22	
October:													
1941	5.99	14.20	13.33	33.53	20.75	6.35	12.99	11.56	51.64	5.24	9.59	14.83	100
1945	4.94	13.04	13.52	31.50	18.30	6.81	12.64	12.04	49.79	6.06	12.65	18.71	100
1945:1941	.82	.92	1.01	.94	.88	1.07	.97	1.04	.96	1.16	1.32	1.26	
November:													
1941	5.78	13.80	13.96	33.54	20.73	6.12	12.80	12.13	51.78	5.21	9.47	14.68	100
1945	4.48	12.39	14.16	31.02	18.47	7.06	11.60	13.30	50.43	5.63	12.91	18.55	100
1945:1941	.78	.90	1.01	.92	.89	1.15	.91	1.10	.97	1.08	1.36	1.26	
December:													
1941	5.30	12.59	14.47	32.36	20.98	6.60	12.61	13.39	53.58	4.67	9.39	14.06	100
1945	4.38	11.41	15.22	31.02	17.83	6.30	10.96	14.19	49.28	5.67	14.04	19.70	100
1945:1941	.83	.91	1.05	.96	.85	.95	.87	1.06	.92	1.21	1.50	1.40	
Total:													
1941	5.75	14.15	13.94	33.83	20.90	6.19	12.55	11.61	51.25	5.21	9.70	14.92	100
1945	4.65	12.42	13.77	30.83	19.05	6.99	12.22	12.93	51.19	5.84	12.13	17.98	100
1945:1941	.81	.88	.99	.91	.91	1.13	.97	1.11	1.00	1.12	1.25	1.20	

TABLE 4.—*Survey period, number of stations operated, number of vehicles counted and weighed in each State in the special weight survey during the summer of 1945*

Region and State	Survey period	Number of stations	Number of vehicles counted			Number of trucks weighed
			All military	All civilian	Civilian trucks	
New England:						
Connecticut	July 23–Aug. 7	10	40	17,693	3,295	1,924
Maine	July 23–Aug. 2	8	41	9,576	2,383	1,512
Massachusetts [1]						
New Hampshire	Aug. 13–Aug. 24	5		8,842	1,091	382
Rhode Island [1]						
Vermont [1]						
Subtotal		23	81	36,111	6,774	3,818
Middle Atlantic:						
New Jersey	Aug. 6–Aug. 13	10	98	38,570	8,971	1,635
New York	Sept. 25–Sept. 28	20	63	21,875	5,749	2,759
Pennsylvania	Aug. 6–Aug. 21	14	134	21,981	4,496	1,135
Subtotal		44	295	82,426	19,216	5,529
South Atlantic:						
Delaware	Sept. 10–Sept. 13	4	97	9,449	2,578	390
Florida [1]						
Georgia	July 17–July 31	10	52	7,519	1,735	1,336
Maryland	July 30–Aug. 10	10	314	20,440	4,393	876
North Carolina	Aug. 6–Aug. 24	10	85	11,056	2,519	2,240
South Carolina [1]						
Virginia [1]						
West Virginia	Aug. 14–Sept. 20	9		6,744	1,741	836
Subtotal		43	548	55,208	12,966	5,678
East sectional subtotal		110	924	173,745	38,956	15,025
East North Central:						
Illinois	Aug. 31–Aug. 29	43	204	49,226	9,329	4,656
Indiana [1]						
Michigan	July 26–Aug. 9	10	59	16,554	2,884	1,245
Ohio [1]						
Wisconsin	Aug. 2–Aug. 23	12	90	14,522	2,589	1,910
Subtotal		65	353	80,302	14,802	7,811
East South Central:						
Alabama	Aug. 15–Sept. 6	10	40	6,407	1,518	838
Kentucky	June 20–July 19	10	141	6,023	1,516	608
Mississippi	Aug. 13–Sept. 6	15	149	13,399	3,172	2,995
Tennessee	July 23–Aug. 3	10	9	4,925	1,509	553
Subtotal		45	339	30,754	7,715	4,994
West North Central:						
Iowa	July 24–Aug. 6	10	32	5,902	1,269	1,218
Kansas	Aug. 16–Aug. 30	10	37	6,109	1,312	1,210
Minnesota	Aug. 6–Aug. 24	10	10	8,050	1,603	1,259
Missouri [1]						
Nebraska	July 26–Aug. 9	11	58	5,741	1,523	1,461
North Dakota	Aug. 2–Aug. 30	10		8,266	2,092	1,435
South Dakota	July 30–Aug. 29	12	24	4,034	889	861
Subtotal		63	161	38,102	8,688	7,444
West South Central:						
Arkansas	July 23–Aug. 3	10	112	8,185	2,593	1,203
Louisiana	July 23–Aug. 3	10	137	5,726	1,555	945
Oklahoma	Aug. 20–Aug. 21	10	110	9,826	2,019	2,019
Texas	July 30–Aug. 24	18	252	18,453	4,117	3,122
Subtotal		48	611	42,190	10,284	7,289
Midwest sectional subtotal		221	1,464	191,348	41,489	27,538
Mountain:						
Arizona	July 16–July 27	10	74	4,471	984	452
Colorado	July 20–Aug. 3	10	199	10,631	2,132	638
Idaho	Aug. 6–Aug. 29	13	49	7,577	1,521	1,283
Montana	Aug. 1–Aug. 28	20	40	8,224	2,008	1,911
Nevada	July 31–Aug. 14	10	95	3,723	665	633
New Mexico	July 23–Aug. 8	10	196	5,805	1,481	1,092
Utah [1]						
Wyoming	Aug. 1–Aug. 14	10	47	3,790	795	717
Subtotal		83	700	44,221	9,587	6,726
Pacific:						
California	Aug. 21–Sept. 21	17	533	58,943	10,634	2,669
Oregon [1]						
Washington	Sept. 5–Sept. 18	10	358	14,445	1,932	1,353
Subtotal		27	891	73,388	12,566	4,022
West sectional subtotal		110	1,591	117,609	22,153	10,748
United States total		441	3,979	482,702	102,598	53,311

[1] Survey not made.

TABLE 5.—*Ratio of 1945 summer counts to corresponding counts in 1944, by regions*

Region	Military vehicles	Civilian vehicles							
		All	Passenger cars			Trucks and truck combinations			Busses
			All	Local	Foreign	All	Single units	Combinations	
Eastern regions:									
New England	0.49	1.20	1.21	1.15	1.40	1.17	1.17	1.17	1.11
Middle Atlantic	.41	1.20	1.24	1.18	1.84	1.08	1.05	1.15	1.16
South Atlantic	.55	1.18	1.24	1.23	1.25	1.03	1.06	.97	1.22
Average	.50	1.19	1.23	1.20	1.40	1.06	1.07	1.04	1.18
Midwest regions:									
East North Central	.68	1.15	1.18	1.21	1.49	1.07	1.06	1.10	.98
East South Central	.65	1.15	1.20	1.21	1.17	1.01	1.03	.96	.93
West North Central	.63	1.18	1.20	1.14	1.67	1.11	1.11	1.10	1.24
West South Central	.82	1.14	1.17	1.09	1.68	1.06	1.07	1.05	1.05
Average	.72	1.16	1.18	1.13	1.50	1.07	1.07	1.07	1.03
Western regions:									
Mountain	1.39	1.24	1.31	1.22	1.58	1.04	1.02	1.13	1.09
Pacific	.37	1.38	1.41	1.36	1.82	1.33	1.31	1.37	.89
Average	.52	1.33	1.37	1.32	1.68	1.23	1.19	1.33	.96
United States average	.59	1.20	1.23	1.18	1.49	1.10	1.09	1.11	1.07

busses were counted but were not stopped. All trucks stopped were weighed and the heavier vehicles were also measured. Information was recorded concerning the type of vehicle whether loaded or empty, number of axles, the manufacturers' rated capacity, and the load on each wheel on one side of the vehicle. The distance between axles of single-unit trucks weighing 13 tons or more and of combinations weighing 17 tons or more also was recorded. The number of vehicles weighed was 53,311 or 52.0 percent of the civilian trucks passing the stations.

TRENDS DETERMINED BY THE SUMMER WEIGHT SURVEY

The ratio of the numbers of the various types of vehicles counted in the 1945 summer survey as compared to the corresponding counts of 1944 are given in table 5 for each region of the country and for the United States. During 1945 there was a large general reduction in military traffic and a recovery of civilian traffic. The increase in military traffic found in the Mountain States was concentrated largely in Colorado and New Mexico.

In table 6 is shown a comparison by census regions of the estimated vehicle-miles traveled on all main rural roads in 1941, 1944, and 1945 by all types of vehicles and by trucks and truck combinations. This table shows the average carried loads and ton-mileage in each of the 3 years, and percentage relation between those years.

The vehicle-mileage of trucks increased in all regions from 1944 to 1945 and ton-mileage increased in all except the East South Central region. For the country as a whole the vehicle-mileage increase amounted to 10 percent and the ton-mileage increase amounted to 13 percent, there being a slight increase in the average weight of the carried load. In the Pacific region the increase was 33 percent for vehicle-miles and 41 percent for ton-miles, reflecting the greater activity in that region during 1945 as war activities increased in the Pacific. There was no significant change in the relation between single-unit trucks and truck combinations except in the two western regions where there was an

TABLE 6.—*Comparison of estimated 1945 vehicle-miles, percentage loaded, average carried load and ton-miles on main rural roads with corresponding figures for 1941 and 1944*

Region and year	All vehicles vehicle-miles	All trucks (including combinations)					Single-unit trucks					Truck combinations				
		Percentage of all vehicles	Vehicle-miles	Percentage loaded	Carried load		Percentage of all trucks and combinations	Vehicle-miles	Percentage loaded	Carried load		Percentage of all trucks and combinations	Vehicle-miles	Percentage loaded	Carried load	
					Average weight	Ton-miles				Average weight	Ton-miles				Average weight	Ton-miles
EASTERN REGION: New England:	*Thousands*		*Thousands*		*Tons*	*Thousands*		*Thousands*		*Tons*	*Thousands*		*Thousands*		*Tons*	*Thousands*
1941	6,663,830	14.7	980,695	72.9	2.79	1,993,120	90.5	887,783	72.7	2.22	1,435,923	9.5	92,912	74.3	8.06	557,197
1944	3,297,741	19.9	656,888	54.8	3.97	1,429,887	81.0	531,951	51.4	2.42	660,579	19.0	124,937	69.3	8.88	769,308
1945	3,928,601	19.6	769,497	53.8	3.92	1,623,664	81.0	623,565	50.9	2.27	721,397	19.0	145,932	66.4	9.31	902,267
1945:1941	.59	1.33	.78	.74	1.41	.81	.90	.70	.70	1.02	.50	2.00	1.57	.89	1.16	1.62
1945:1944	1.19	.98	1.17	.98	.99	1.14	.90	1.17	.99	.94	1.09	1.00	1.17	.96	1.05	1.17
Middle Atlantic:																
1941	17,108,107	20.2	3,447,631	70.3	3.41	8,267,429	81.0	2,793,416	69.9	2.14	4,175,576	19.0	654,215	72.1	8.67	4,091,853
1944	9,010,063	24.1	2,171,217	55.8	4.14	5,014,668	75.8	1,646,177	53.5	2.35	2,073,208	24.2	525,040	63.1	8.87	2,941,460
1945	10,725,475	21.8	2,337,486	54.6	4.30	5,487,126	74.2	1,735,098	51.9	2.36	2,129,030	25.8	602,388	62.3	8.95	3,358,096
1945:1941	.63	1.08	.68	.78	1.26	.66	.92	.62	.74	1.10	.51	1.36	.92	.86	1.03	.82
1945:1944	1.19	.90	1.08	.98	1.04	1.09	.98	1.05	.97	1.00	1.03	1.07	1.15	.99	1.04	1.14
South Atlantic:																
1941	18,612,782	23.2	4,315,146	65.6	3.88	10,982,765	77.8	3,357,800	64.6	2.61	5,663,491	22.2	957,346	69.0	8.05	5,319,274
1944	10,978,795	26.6	2,917,279	58.9	4.85	8,345,945	68.7	2,002,776	55.2	2.52	2,782,538	31.3	914,503	67.2	9.06	5,563,407
1945	12,835,292	23.4	3,007,162	57.4	4.84	8,350,393	70.6	2,121,565	53.2	2.48	2,803,119	29.4	885,597	67.5	9.28	5,547,274
1945:1941	.69	1.01	.70	.88	1.25	.76	.91	.63	.82	.95	.49	1.32	.93	.98	1.15	1.04
1945:1944	1.17	.88	1.03	.97	1.00	1.00	1.03	1.06	.96	.98	1.01	.94	.97	1.00	1.02	1.00
Subtotal:																
1941	42,384,719	20.6	8,743,472	68.2	3.56	21,243,314	80.5	7,038,999	67.7	2.37	11,274,990	19.5	1,704,473	70.5	8.30	9,968,324
1944	23,286,599	24.7	5,745,384	57.3	4.49	14,790,500	72.8	4,180,904	54.0	2.44	5,516,325	27.2	1,564,480	66.0	8.98	9,274,175
1945	27,489,368	22.2	6,114,145	55.9	4.53	15,461,183	73.3	4,480,228	52.4	2.41	5,653,546	26.7	1,633,917	65.4	9.17	9,807,637
1945:1941	.65	1.08	.70	.82	1.27	.73	.91	.64	.77	1.02	.50	1.37	.96	.93	1.10	.98
1945:1944	1.18	.90	1.06	.98	1.01	1.05	1.01	1.07	.97	.99	1.02	.98	1.04	.99	1.02	1.06
MIDWEST REGION: East North Central:																
1941	24,192,260	17.8	4,305,309	68.7	4.05	11,961,648	68.0	2,926,163	66.0	2.10	4,064,158	32.0	1,379,146	74.3	7.71	7,897,490
1944	13,944,387	19.7	2,743,759	61.2	4.86	8,171,775	63.7	1,746,699	52.8	2.13	1,967,162	36.3	997,060	76.1	8.18	6,204,613
1945	16,018,709	18.4	2,940,681	58.7	5.15	8,897,279	62.9	1,848,320	49.4	2.38	2,176,246	37.1	1,092,361	74.4	8.27	6,721,033
1945:1941	.66	1.03	.68	.85	1.27	.74	.92	.63	.75	1.13	.54	1.16	.79	1.00	1.07	.85
1945:1944	1.15	.93	1.07	.96	1.06	1.09	.99	1.06	.94	1.12	1.11	1.02	1.10	.98	1.01	1.08
East South Central:																
1941	7,533,991	21.7	1,632,240	64.8	3.13	3,303,537	90.0	1,469,033	64.1	2.70	2,546,285	10.0	163,207	70.5	6.58	757,252
1944	5,030,873	22.2	1,118,954	50.4	4.19	2,364,060	81.7	913,756	46.3	2.90	1,225,738	18.3	205,198	68.7	8.08	1,138,322
1945	5,423,111	20.9	1,134,490	47.0	4.01	2,136,854	82.7	938,635	43.4	2.72	1,108,602	17.3	195,855	63.8	8.23	1,028,252
1945:1941	.72	.96	.70	.73	1.28	.65	.92	.64	.68	1.01	.44	1.73	1.20	.90	1.25	1.36
1945:1944	1.08	.94	1.01	.93	.96	.90	1.01	1.03	.94	.94	.90	.95	.95	.93	1.02	.90
West North Central:																
1941	15,260,119	20.8	3,172,121	67.0	3.23	6,872,720	80.8	2,563,019	66.3	2.09	3,549,465	19.2	609,102	70.1	7.79	3,323,255
1944	8,677,255	24.6	2,135,475	61.0	3.41	4,433,066	78.0	1,666,466	59.9	1.94	1,963,870	22.0	469,009	64.7	8.14	2,469,196
1945	10,180,874	23.3	2,367,392	58.8	3.67	5,111,866	78.3	1,853,832	55.8	1.98	2,053,160	21.7	513,560	69.5	8.57	3,058,706
1945:1941	.67	1.12	.75	.88	1.14	.74	.97	.72	.84	.95	.58	1.13	.84	.99	1.10	.92
1945:1944	1.17	.95	1.11	.96	1.08	1.15	1.00	1.11	.93	1.02	1.05	.99	1.09	1.07	1.05	1.24
West South Central:																
1941	13,745,126	21.8	2,994,305	60.6	2.99	5,427,182	78.9	2,362,360	59.6	2.23	3,144,368	21.1	631,945	64.5	5.60	2,282,814
1944	9,445,823	25.7	2,426,267	44.6	3.78	4,085,999	73.7	1,789,322	39.6	2.47	1,746,957	26.3	636,945	58.5	6.28	2,339,042
1945	10,724,349	24.1	2,583,820	42.6	3.79	4,165,546	74.1	1,914,077	37.1	2.54	1,808,167	25.9	669,743	58.2	6.05	2,357,379
1945:1941	.78	1.11	.86	.70	1.27	.77	.94	.81	.62	1.14	.58	1.23	1.06	.90	1.08	1.03
1945:1944	1.14	.94	1.06	.96	1.00	1.02	1.01	1.07	.94	1.03	1.04	.98	1.05	.99	.96	1.01
Subtotal:																
1941	60,731,496	19.9	12,103,975	65.7	3.47	27,565,087	77.0	9,320,575	64.2	2.22	13,304,276	23.0	2,783,400	70.9	7.23	14,260,811
1944	37,098,338	22.7	8,424,455	54.9	4.12	19,054,900	72.6	6,116,243	49.9	2.21	6,903,727	27.4	2,308,212	68.2	7.71	12,151,173
1945	42,347,043	21.3	9,026,383	52.6	4.48	20,311,545	72.6	6,554,864	46.8	2.33	7,146,175	27.4	2,471,519	68.1	7.82	13,165,370
1945:1941	.70	1.07	.75	.80	1.29	.74	.94	.70	.73	1.05	.54	1.19	.89	.96	1.08	.92
1945:1944	1.14	.94	1.07	.96	1.09	1.07	1.00	1.07	.94	1.05	1.04	1.00	1.07	1.00	1.01	1.08
WESTERN REGION: Mountain:																
1941	6,741,489	19.5	1,316,246	62.6	3.39	2,795,644	88.0	1,158,270	61.6	2.46	1,755,379	12.0	157,976	69.7	9.45	1,040,265
1944	4,181,810	25.8	1,078,077	44.2	4.52	2,170,648	84.7	912,870	39.4	2.60	909,968	15.3	165,207	70.8	10.66	1,260,680
1945	5,194,915	21.5	1,118,716	45.8	4.63	2,374,966	83.3	931,905	41.5	2.51	950,677	16.7	186,811	71.5	10.66	1,424,289
1945:1941	.77	1.10	.85	.73	1.37	.85	.95	.80	.67	1.02	.54	1.39	1.18	1.03	1.13	1.37
1945:1944	1.24	.83	1.04	1.04	1.02	1.09	.98	1.02	1.05	.97	1.00	1.09	1.13	1.01	1.00	1.13
Pacific:																
1941	12,647,141	16.0	2,021,200	68.8	5.13	7,132,737	76.1	1,538,563	65.2	2.14	2,151,962	23.9	482,637	80.4	12.84	4,980,775
1944	8,073,814	24.6	1,986,703	63.7	6.85	8,680,071	62.9	1,249,258	54.1	2.45	1,657,480	37.1	737,445	80.1	11.89	7,022,591
1945	10,761,149	15.3	2,647,932	65.4	7.05	12,216,924	61.7	1,634,744	58.1	2.57	2,435,956	38.3	1,013,188	77.3	12.49	9,780,968
1945:1941	.85	.96	1.31	.95	1.37	1.71	.81	1.06	.89	1.20	1.13	1.60	2.10	.96	.97	1.96
1945:1944	1.33	.62	1.33	1.03	1.03	1.41	.98	1.27	1.07	1.05	1.47	1.03	1.37	.96	1.05	1.39
Subtotal:																
1941	19,388,630	17.2	3,337,446	66.4	4.48	9,928,381	80.8	2,696,833	63.7	2.27	3,907,341	19.2	640,613	77.7	12.09	6,021,040
1944	12,255,624	25.0	3,064,780	56.9	6.22	10,850,719	70.5	2,162,128	47.9	2.48	2,567,448	29.5	902,652	78.4	11.70	8,283,271
1945	15,956,064	23.6	3,766,648	59.6	6.50	14,591,890	68.1	2,566,649	51.8	2.55	3,386,633	31.9	1,199,999	76.4	12.22	11,205,257
1945:1941	.82	1.37	1.13	.90	1.45	1.47	.84	.95	.81	1.12	.87	1.66	1.87	.98	1.01	1.86
1945:1944	1.30	.94	1.23	1.05	1.05	1.34	.97	1.19	1.08	1.03	1.32	1.08	1.33	.97	1.04	1.35
UNITED STATES TOTAL:																
1941	122,504,845	19.7	24,184,893	66.7	3.64	58,736,782	78.8	19,056,407	65.4	2.29	28,486,607	21.2	5,128,486	71.6	8.23	30,250,175
1944	72,640,561	23.7	17,234,619	56.1	4.63	44,696,119	72.3	12,459,275	50.9	2.36	14,987,500	27.7	4,775,344	69.4	8.96	29,708,619
1945	85,792,475	22.0	18,907,176	55.1	4.84	50,364,618	71.9	13,601,741	49.6	2.40	16,186,354	28.1	5,305,435	69.2	9.31	34,178,264
1945:1941	.70	1.12	.78	.83	1.33	.86	.91	.71	.76	1.05	.57	1.33	1.03	.97	1.13	1.13
1945:1944	1.18	.93	1.10	.98	1.05	1.13	.99	1.09	.97	1.02	1.08	1.01	1.11	1.00	1.04	1.15

TABLE 7.—*Average maximum axle load of loaded trucks in the summer of 1945 and in corresponding periods of 1944 and 1942 and in the prewar years between 1936 and 1941*

[In pounds]

Region	All trucks and combinations				Single-unit trucks				Truck combinations			
	1945	1941	1942	Prewar	1945	1944	1942	Prewar	1945	1944	1942	Prewar
New England	10,005	10,221	9,310	7,647	7,990	8,372	7,977	6,985	16,621	15,758	14,605	13,951
Middle Atlantic	11,584	11,318	10,865	7,858	9,346	9,118	8,938	6,779	16,768	16,769	16,027	13,551
South Atlantic	9,950	10,246	9,146	8,488	7,632	8,070	7,206	7,589	15,403	15,709	14,416	11,8[illegible]
East North Central	11,678	11,103	10,259	8,290	8,595	8,085	7,829	6,806	14,797	14,659	13,789	12,090
East South Central	9,024	9,372	8,325	7,215	8,003	8,152	7,414	6,962	14,157	13,001	11,732	8,899
West North Central	9,300	9,218	8,798	7,593	7,212	7,167	7,019	6,565	14,633	14,118	13,560	11,594
West South Central	7,673	7,717	7,441	5,868	5,834	5,798	5,422	5,134	14,297	11,981	12,050	8,391
Mountain	8,858	8,551	8,225	6,051	7,110	7,118	6,785	5,444	14,118	13,763	13,896	11,284
Pacific	9,995	10,207	8,980	7,094	8,497	8,668	7,344	5,967	13,504	13,746	13,355	11,176
United States average	10,017	9,967	9,160	7,552	7,869	7,857	7,294	6,566	14,615	14,848	13,788	11,449

appreciable increase in the usage of the heavier type of vehicle.

The comparison between the data for 1945, the last year of the war, and those for 1941, the last prewar year, shows that truck vehicle-mileage dropped 22 percent and that the percentage of loaded vehicles decreased from 66.7 to 55.1, but that notwithstanding this the ton-mileage hauled dropped only 14 percent. The reason for the comparatively small drop in ton-mileage was a 33 percent increase in the average weight of carried load, due largely to the increased proportion of heavy truck combinations in the total number of trucks. In 1941, 21.2 percent of the cargo vehicles on the road were combinations, and in 1945 this percentage had increased to 28.1. There was also some increase in the average weight of the load carried by both truck combinations and single-unit trucks.

FREQUENCY OF HEAVY LOADS INCREASED

Table 7 shows the average maximum axle load of loaded vehicles in the summer of 1945 and in corresponding periods of previous years when such data were collected. Axle loads in 1945 were substantially the same as in 1944 but were much heavier on the average than in prewar years. For the United States as a whole the average maximum axle load increased from 7,552 pounds in the prewar period to 10,017 in 1945.

While the average maximum axle loads increased during the war, frequency of the heavier axle loads also increased. The increase in the frequency of these heavier loads is shown in table 8. It will be noted that in each weight group for which data are included in the table there was an increase in the frequency of the heavier loads, and that in each of the three cases the frequency in 1945 was about five times the frequency found in the original prewar survey. The highest frequency of heavy axle loads is found in the Middle Atlantic region with the second highest frequency in the New England area. Other regions are far behind these two.

The frequency of heavy vehicles is shown in table 9. It will be observed that the heavier units have become more frequent throughout the war and in 1945 were far more frequent than in the prewar period. For example, of each 1,000 trucks weighed in the survey made before the war only three were as heavy as 50,000 pounds, while in 1945 the proportion of trucks this heavy amounted to twenty-three per 1,000. These extremely heavy vehicles are found principally in the Pacific region, but are on the increase throughout the country.

In limiting loads to prevent overstressing bridges both the spacing of axles and the actual axle loads should be taken into account in determining the permissible gross load. The so-called gross load formula has been widely used to indicate permissible combinations of axle loads and spacings. The formula referred to is $W=C\,(L+40)$ in which W is the total weight of the vehicle in pounds, or the weight of an interior group of axles, and L is the distance in feet between the first

TABLE 8.—*Number of axle loads of 18,000, 20,000, and 22,000 pounds or more per 1,000 loaded and empty trucks and combinations in the summers of 1945, 1944, 1942, and in the prewar period between 1936 and 1941*

Region	Number of heavy axle loads per 1,000 loaded and empty trucks and combinations of—											
	18,000 pounds or more				20,000 pounds or more				22,000 pounds or more			
	1945	1944	1942	Prewar	1945	1944	1942	Prewar	1945	1944	1942	Prewar
New England	109	108	65	18	62	55	26	21	29	25	12	8
Middle Atlantic	119	138	91	40	75	72	45	18	33	48	18	7
South Atlantic	78	64	45	7	26	23	14	4	7	9	4	[1] 0
East North Central	74	70	48	14	10	17	11	4	5	5	3	2
East South Central	30	29	14	1	15	11	7	[1] 0	6	3	4	[1] 0
West North Central	41	37	29	5	8	7	6	[1] 0	1	1	1	[1] 0
West South Central	25	28	18	4	8	9	4	2	2	5	2	[1] 0
Mountain	44	20	24	5	12	6	5	3	3	2	1	2
Pacific	47	62	24	3	13	17	4	1	3	3	2	[1] 0
United States average	67	64	41	13	23	23	13	5	9	11	5	2

[1] Less than 5 per 10,000 counted.

TABLE 9.—*Number of gross weights of 30,000, 40,000, and 50,000 pounds or more per 1,000 loaded and empty trucks and combinations in the summers of 1945, 1944, 1942, and in the prewar period between 1936 and 1941*

Region	Number of gross weights per 1,000 vehicles—											
	30,000 pounds or more				40,000 pounds or more				50,000 pounds or more			
	1945	1944	1942	Prewar	1945	1944	1942	Prewar	1945	1944	1942	Prewar
New England	113	120	97	58	50	51	30	15	10	6	3	1
Middle Atlantic	163	172	142	62	77	70	48	17	18	16	12	3
South Atlantic	133	120	97	32	43	32	21	3	2	3	1	[1] 0
East North Central	233	221	175	65	72	58	40	13	28	21	10	5
East South Central	57	63	39	7	12	10	2	1	1	[1] 0	1	[1] 0
West North Central	114	110	91	34	33	27	16	2	11	7	1	[1] 0
West South Central	73	62	59	8	20	14	8	1	3	2	1	[1] 0
Mountain	98	80	76	20	51	42	41	7	28	22	21	3
Pacific	156	170	155	97	116	109	110	47	86	83	81	24
United States average	144	134	111	43	58	47	33	11	23	19	12	3

[1] Less than 5 per 10,000 counted.

TABLE 10.—*Number of trucks with values of C [1] in the gross weight formula in excess of various values, per 1,000 loaded and empty trucks and combinations in the summers of 1945, 1944, and 1942*

Region	Number of trucks and combinations with values of C—																	
	Over 650			Over 700			Over 750			Over 800			Over 850			Over 900		
	1945	1944	1942	1945	1944	1942	1945	1944	1942	1945	1944	1942	1945	1944	1942	1945	1944	1942
New England	33	35	21	18	18	11	9	8	5	4	4	2	2	1	1	1	1	0
Middle Atlantic	56	48	35	33	31	23	18	20	13	9	11	8	5	4	4	2	2	2
South Atlantic	20	22	9	9	10	4	5	5	2	3	2	1	1	1	[2] 0	1	[2] 0	[2] 0
East North Central	48	33	26	37	23	16	28	15	11	22	11	7	14	6	5	8	4	2
East South Central	5	5	2	1	3	1	[2] 0	1	[2] 0	[2] 0	[2] 0	[2] 0	[2] 0	[2] 0	[2] 0	[2] 0	[2] 0	[2] 0
West North Central	21	13	9	13	6	3	8	2	1	3	1	[2] 0	2	[2] 0	[2] 0	1	[2] 0	[2] 0
West South Central	9	10	6	3	3	3	1	1	2	1	1	1	[2] 0	1	1	[2] 0	[2] 0	[2] 0
Mountain	45	34	41	34	25	31	22	16	21	16	10	15	11	7	11	6	3	8
Pacific	135	87	140	112	74	97	89	56	82	59	33	54	22	11	21	8	3	13
United States average	45	32	22	33	22	15	24	14	13	17	9	8	8	3	4	4	1	2

[1] Derived from the gross load formula $W = C(L+40)$ in which W is the total weight of the vehicle in pounds, or the weight of an interior group of axles, and L is the distance in feet between the first and last axle of the vehicle, or of any interior group of axles.
[2] Less than 5 per 10,000.

and last axle of the vehicle, or of any interior group of axles. C is a measure of the load concentration and it is generally thought that a value of C greater than 750 is excessive. Thus the trend in the frequency of C values above 750 may be used as an index of the trend in the practice of excessive loading.

Table 10 gives the frequency of trucks and combinations with various values of C in 1945, 1944, and 1942. Heavier load concentrations were most frequent in the Pacific region with the Middle Atlantic region second. There was a steady increase in the frequency of heavy load concentration and in each of the six groups shown in the table the heavy concentrations were approximately twice as frequent in 1945 as in 1942.

(*Referred from p. 252*)

APPENDIX.—SPECIFICATIONS FOR COTTON FABRIC FOR USE IN ROAD CONSTRUCTION

Cotton fabric used for the reinforcement of bituminous surface treatments shall meet the following specifications.

GENERAL REQUIREMENTS OF THE FABRIC

The fabric for use in bituminous surface treatments shall be free from avoidable imperfections of manufacture or other defects which may affect its appearance or serviceability.

RAW MATERIAL REQUIREMENTS

The fabric shall be made of raw cotton, cotton waste, or mixtures of raw cotton and cotton waste, from cotton grown and/or manufactured in the United States, of sufficient quality to obtain the fabric strength and other requirements of serviceability indicated herein.

YARN REQUIREMENTS

(*a*) Ply—the yarn shall be two-ply.
(*b*) Twist—the twist in the two-ply yarn shall be such as to give what is known in the trade as a "balanced twist."
(*c*) Sizing—no sizing shall be applied to the yarn.
(*d*) Count and Strength—the yarn shall have a count and strength sufficient to make a fabric which will meet the specifications.

DETAILED REQUIREMENTS OF THE FABRIC

(*a*) Weave—the weave shall be plain.
(*b*) Sizing—no sizing shall be applied to the fabric.
(*c*) Length of Rolls—a roll shall consist of a "cut" not shorter than 40 yards and not longer than 120 yards.
(*d*) Construction, etc.—the thread count, weight, width, and breaking strength of the various types of fabrics shall be as shown in table 1, except as noted in the tolerances.

TOLERANCES

(*a*) Thread Count—a total plus or minus tolerance of two threads per 5 inches in a combination of warp and filling will be permitted.
(*b*) Width—a tolerance of plus or minus 1 inch from the width specified will be permitted.
(*c*) Breaking Strength—the combined strength of the warp and filling in the fabric shall equal or exceed the combined strength of these elements given in the above table and neither element shall be more than 10 percent under the requirements.
(*d*) Weight—a tolerance of plus or minus 5 percent will be permitted.

TABLE 1.—*Construction specifications for cotton fabric for use in road construction*

Designation	Weight per square yard	Thread count per inch		Width [1]	Minimum average breaking strength (grab method)	
		Warp	Filling		Warp	Filling
	Ounces			*Inches*	*Pounds*	*Pounds*
A–1	5.30	12	12	90	45	45
A–2	5.30	12	12	82	45	45
A–3	5.30	12	12	74	45	45
B–1	4.25	9	9	90	35	35
B–2	4.25	9	9	82	35	35
B–3	4.25	9	9	74	35	35
C–1	3.20	7	7	90	25	25
C–2	3.20	7	7	82	25	25
C–3	3.20	7	7	74	25	25

[1] The widths of 74, 82, and 90 inches are intended for use in surfaces 18, 20, and 22 feet wide, respectively, thus providing that the surface will be covered with three strips of fabric with laps of approximately 3 inches.

METHODS OF SAMPLING AND TESTING

(*a*) Sampling—not less than one sample, at least 1 yard in length and the full width of the fabric, shall be taken at random from each 1,000 yards or fraction thereof, except when the shipment is over 10,000 yards, in which case at least one sample shall be taken from each one-tenth of the shipment.
(*b*) Testing—Federal Specification CCC–T–191, Textiles; Test Methods, of the issue in effect on date of invitation for bids, wherever practicable, shall be followed.

PACKING AND MARKING

(*a*) Packing—the fabric shall be shipped in rolls not to exceed 120 yards per roll and shall be covered with material so as to insure acceptance by common or other carrier, for safe transportation, at the lowest rate, to the point of delivery.
(*b*) Marking—unless otherwise specified, shipping containers shall be marked with the name of the material, the style, the width, the quantity contained therein, the name of the contractor, and the number of the order.

AERIAL SURVEYING ON THE ALASKA HIGHWAY, 1942

Reported by WILLIAM T. PRYOR, Highway Engineer,[1] Department of Design, Public Roads Administration

THE ALASKA HIGHWAY crosses what was in 1942 perhaps the largest unmapped wilderness left on the North American Continent. The highway begins in Canada at the end of the 495-mile railroad reaching north and west from Edmonton, Alberta, to Dawson Creek in British Columbia. From Dawson Creek the highway extends in a northwesterly direction through British Columbia and Yukon Territory into Alaska where, at Big Delta, it joins the Richardson Highway, an existing road leading northwest about 100 miles to Fairbanks and southerly to Valdez. (See fig. 1.)

Work on the highway began in 1942 at a time when the Japanese were gaining a foothold in the Aleutian Islands and the American Continent was thought to be in danger of invasion by way of Alaska.

The agreement [2] between Canada and the United States called for the construction of a highway along a route joining the airports at Fort St. John, Fort Nelson, Watson Lake, Whitehorse, Boundary, and Big Delta; the respective termini connecting with existing roads in Canada and Alaska. The airports had been located at intervals along the best all-year flying route across the vast subarctic region between the principal centers in Canada and the United States and Fairbanks.

Completion of the highway in the shortest possible time was imperative, in the face of innumerable obstacles. Difficulties arose at the very beginning, for both the Public Roads Administration and the Army Corps of Engineers discovered that there did not exist any detailed maps showing the topography of the vast area through which a route had to be found—an expanse 1,500 miles long and 100 to 250 miles wide, comprising about one-quarter million square miles in the subarctic region.

The only maps available, covering the headwaters of the Mackenzie and Yukon Rivers, lacked planimetric detail and accuracy. They were at scales of 1:100,000 to 1:2,000,000 and even smaller, and little could be learned from them about the topography and general character of the area.

Trappers, Indians, and "bush" pilots, and others who were likely to be familiar with any part of the area along the projected route, were interviewed. The descriptions they gave only confirmed what was already known—that there would be found areas of muskeg (sphagnum swamp), high mountain ranges, permanently frozen soil, dense forests which everywhere obstruct the view and hinder the traveler, countless lakes, and numerous rivers which would be difficult to cross.

The airport locations, known by latitude and longitude, and the small-scale maps showing general locations of rivers and mountain ranges, were all the engineers had in the beginning for guidance. The 1,423-mile project was split into two parts: The southern division, with headquarters at Fort St. John, extended 634 miles from Dawson Creek to a point near Watson Lake; the northern division continued from that point for 789 miles to Big Delta, and had its headquarters at Whitehorse (fig. 1).

FIGURE 1.—MAP OF THE ALASKA HIGHWAY.

TRANSPORTATION DIFFICULTIES ENCOUNTERED

The problem of locating a highway in unmapped country is rarely encountered in the United States. As implied in the word "unmapped," there was almost complete absence of roads or even usable trails in the wilderness region through which the Alaska Highway was built. Surveying and mapping parties had little to guide them.

Transportation of men and supplies and equipment was always difficult. At the southern end of the highway a provincial road led northerly 60 miles from Dawson Creek to Fort St. John, and an old winter road which could be traveled only when the ground was frozen continued to Fort Nelson. This winter road is shown in figure 2, winding across the fairly level terrain beyond the east bank of the Prophet River.

North of Fort Nelson, overland travel was accomplished by primitive methods. Dog teams were used while snow remained on the ground and rivers and lakes were frozen over, but in summer the men often pushed forward on foot, carrying their supplies on their backs. Pack and saddle horses were sometimes used, but they were not easy to obtain and it was difficult to get them into the back country and to feed them there.

Motorboats and small wood-burning steamers were used during the summer wherever the route lay along the larger rivers and chains of lakes. A number of ferries were also placed in operation at river crossings.

[1] The author was on the staff of the construction engineer for the southern division of the Alaska Highway and was in charge of design during the period of operations covered by this article.

[2] Executive Agreement Series 246 of the Department of State.

FIGURE 2.—AN OLD WINTER ROAD CONNECTING FORT ST. JOHN AND FORT NELSON. ICE ON THE RIVERS AND LAKES CAN BE SEEN IN THIS AERIAL VIEW.

Only land-based airplanes flying from the existing airfields, or float planes flying from lakes or rivers (ski planes were used in winter) were free to come and go at will. Even they were handicapped at times by limited fuel supplies and bad weather. Aircraft were used to transport the field parties and their equipment to advance bases whenever possible.

HARSH CLIMATE FACED

The average annual temperature along the highway ranges from 35° Fahrenheit at Fort St. John to 26° at Fairbanks, as compared to 50° or more in the United States. Temperatures range from a high of 105° to 76° below zero. The winters are cold, long, and dry;

spring and fall are very short; and summer, from early June to September, is a cool wet season.

Daylight in midwinter lasts about 6 hours. Subzero temperatures prevail for weeks at a time, to be broken by chinook winds which raise temperatures to above 40° for a few days.

In summer there are as many as 20 daylight hours, and nights are short twilight periods. During early summer the midges, biting flies, and mosquitoes are extremely annoying pests.

Housing of survey crews was always a difficult problem, and sometimes men had to camp in the open. The tents (fig. 3) which the field parties often used for quarters, kitchen, mess hall, and offices were not comfortable nor did they afford complete protection under even usual climatic conditions of the region.

FIGURE 3.—A TENT CAMP WAS HOME AND OFFICE FOR SURVEY CREWS.

TOPOGRAPHY A MAJOR OBSTACLE

Some idea of the nature of the country through which the Alaska Highway was located and built can be obtained from the illustrated examples that follow. For descriptive purposes the various kinds of topography are grouped in three classes, according to the difficulty they presented to the highway engineer.

Impassable topography.—In impassable topography a road suitable for all-year use could not be built in a reasonable length of time. Figure 4, showing a section of the Canadian Rockies about 100 miles southwest of Fort Nelson, illustrates this type of topography.

Difficult topography.—In this kind of topography special problems are encountered in locating and building a road. In the area north of the Canadian Rockies, shown in figure 5, A, the ground is fairly level as indicated by the numerous small lakes and the ox bows in the rivers; but there are many areas of muskeg, composed of water-saturated muck, fine silty soil, and decaying vegetation with a cover of moss, grasses, willows, and low-growing bushes. Stunted evergreens often grow scatteringly on these areas. Muskeg is an extremely

FIGURE 4.—IMPASSABLE TOPOGRAPHY—HIGH MOUNTAINS AND GLACIERS.

FIGURE 5.—DIFFICULT TOPOGRAPHY—(A) MUSKEG ON LEVEL GROUND, AND (B) A NARROW, TORTUOUS RIVER VALLEY.

unstable base for road building. It occurs not only in valleys and on level ground, but also on hillsides wherever water is retained. The subsoil underlying muskeg is always impermeable and frequently is permanently frozen (permafrost).

A different type of difficult topography is shown in figure 5, B, a photograph of the Trout River valley near its confluence with the Liard. The river flows north out of the Canadian Rockies through rugged ground. The highway was built along this river valley, and tortuous alinement and steep grades were necessary. Water seeping and flowing down the slopes under a cover of vegetal debris caused slides in spring and summer and formation of ice in winter.

Comparatively easy topography.—Figure 6 shows comparatively easy topography in an area north of the Canadian Rockies where the ground is level to rolling. The pioneer road followed an easy route on the near side of the Liard River. Eskers and drumlins, left by the receding glaciers, are a definite indication of the presence of granular soils. Eskers are long narrow ridges of gravel or sand and silt deposited by subglacial streams. Drumlins are elongate or oval hills of glacial drift.

PURPOSE OF THE PIONEER ROAD

Plans for construction of the Alaska Highway required that a route connecting the airports designated as control points be selected by the Public Roads Administration. A pioneer road was to be built by Army Engineer troops following the line selected for the final road as closely as might be practicable. This road was to be used for movements necessary in constructing the final highway.

Actually, location of the route and construction of the pioneer road began almost simultaneously and progressed concurrently throughout the construction season of 1942. Much of the pioneer road was located before the exact line of the final road had been fixed.

Construction crews were soon close at the heels of the survey parties. At the very beginning it became evident that usual methods of ground reconnaissance

and survey would not suffice. The only hope of keeping up with the work schedule established to meet the war emergency lay in the use of aerial methods which permitted seeing and studying the topography far in advance of construction. Aerial methods enabled the engineers to examine the unexplored territory much more quickly and fully than was possible by ground reconnaissance, and without the difficulties and delays which hampered ground operations.

EVOLUTION OF AERIAL SURVEYING

The use of aircraft and aerial photography in highway location has passed through progressive stages of evolution. At first aircraft were used for observation flights over wide areas, much as reconnaissance parties might scout the topography on the ground. A logical advance was the taking of aerial oblique photographs: Bird's-eye views showing topography in perspective, much as the aerial observer sees it.

Then came the practice of taking vertical photographs in series along a predetermined line of flight. These photographs can be matched and assembled in a mosaic to form a composite picture of the area photographed. Next followed the application of the stereoscopic principle, using successive pairs of vertical photographs to see the topography in three dimensions.

The most recent development in aerial surveying, as applied to highway work, is the making of accurate large-scale contour maps from vertical photographs that have been adjusted to controls established by ground surveys.

All of these methods may be combined in their proper sequence for use in highway location. On the Alaska Highway all methods except the making of contour maps were used. Because of the haste which the exigencies of war required and the inexperience in aerial survey methods of most of the location engineers, full advantage could not always be taken of the aerial survey methods. Sometimes the survey parties, hard pressed by the pioneer road construction crews, could not wait for the taking of needed additional aerial photographs. Where the photographs were available in time, and as the engineers gained experience, the aerial survey procedure was always found to be a time-saving means of locating the most practicable route for the pioneer road.

RECONNAISSANCE BY AERIAL OBSERVATION

Observation flights for highway reconnaissance are generally planned from available topographic maps, but for the Alaska Highway no suitable maps existed. The flights were therefore made along compass courses between the airfields, whose positions had been determined. Lakes, rivers, mountain peaks and passes, and identifiable muskeg were observed on these flights; and the more prominent landmarks were selected as guides for aerial photographic flights and for the ground survey crews.

While akin to field reconnaissance, the aerial observation flights made it possible to see in a few hours an expanse of area which, in wilderness such as that along the Alaska Highway, could be examined on the ground only by weeks or months of arduous travel. Densely timbered areas, which were impossible to study on the ground except for occasional glimpses from a hill or treetop, could be scanned easily from the air.

OBSERVATION RECORDED BY OBLIQUE PHOTOGRAPHS

The Army Air Corps during the fall of 1941 took many aerial oblique photographs over a wide area between the airfields in Canada. These photographs were taken from an average elevation of 20,000 feet with a camera having a 6-inch focal length lens. Generally the photographs were high-obliques, so-called because the horizon appears in the picture.

Figure 7 is a high-oblique photograph of the Peace River near its emergence from the Canadian Rockies west of Hudson's Hope. Study of such photographs along the river showed that the valley might have been a feasible route for the highway, but the final location was chosen to follow the line of airfields.

A valuable development evolved from the taking of high-obliques is trimetrogon photography, producing left and right high-obliques combined with a vertical between them. A set of photographs, taken simultaneously by three 6-inch lens cameras from an altitude of 20,000 feet, provides horizon-to-horizon coverage. The effectively useful portions of the photographs cover an area 25 to 50 miles wide. In figure 8 a set of trimetrogon photographs, matched together, shows a mountain divide between two broad river valleys. Study of trimetrogon photographs taken along this divide would aid in choosing a pass between the river valleys for more detailed aerial examination. The Air Corps took many of these trimetrogon photographs as well as single high-obliques.

The oblique photographs recorded for further use what the observers had seen during their reconnaissance

FIGURE 6.—COMPARATIVELY EASY TOPOGRAPHY—ROLLING GROUND WITH GLACIAL DEPOSITS.

FIGURE 7.—THE HIGH-OBLIQUE PHOTOGRAPH GIVES A PERSPECTIVE PICTURE.

FIGURE 8.—TRIMETROGON PHOTOGRAPHS HAVE HORIZON-TO-HORIZON COVERAGE.

FIGURE 9.—A LARGE-SCALE VERTICAL PHOTOGRAPH; SCALE 4 INCHES PER MILE.

flights. Possible routes for the highway could be selected from them and impassable obstacles avoided. Flight lines for taking strips of vertical photographs were planned along tentatively chosen routes and the pilots were guided on such flights by identifiable landmarks on the obliques.

DETAILED STUDIES MADE WITH VERTICAL PHOTOGRAPHS

During the spring and summer of 1942 the Army Air Corps took aerial vertical photographs on flight lines along many of the possible routes observed in aerial reconnaissance or on the high-oblique photographs.

The verticals were taken with a 6-inch lens camera, generally from an altitude of 10,000 feet. Taking into account the average elevation of the topography, such vertical photographs had a scale of about 4 inches per mile. Figure 9, showing an area along the Muskwa River, is a vertical at this scale. Slide areas and drainage channels can be seen clearly; even individual trees can be recognized.

Vertical photographs of considerably smaller scale were also taken, but were of much less value in locating the pioneer road. Figure 10 is a vertical of the area at the confluence of the Liard and Turnagain Rivers, at a scale of about 2 inches per mile. The general character of the topography can be seen, but much of the detail needed for location study is not visible.

Figure 11 shows the locations of some of the vertical photographs taken on flights along possible routes chosen for detailed study in the area south and west of Fort Nelson—only every other photograph of each flight line is represented, and some flight lines are omitted altogether, in order to avoid crowding. The overlaps and gaps in the flight lines reflect the haste with which the photography was necessarily done, and the inability to plan the work completely in advance because of the lack of suitable maps such as would be available for similar work in the United States.

Uncontrolled mosaics were made simply by matching the conjugate images of adjacent vertical photographs, as shown in figure 12. The pioneer road had already been pushed through this area, and appears as a white line on the edges of some of the photographs. An uncontrolled mosaic made from verticals taken on several

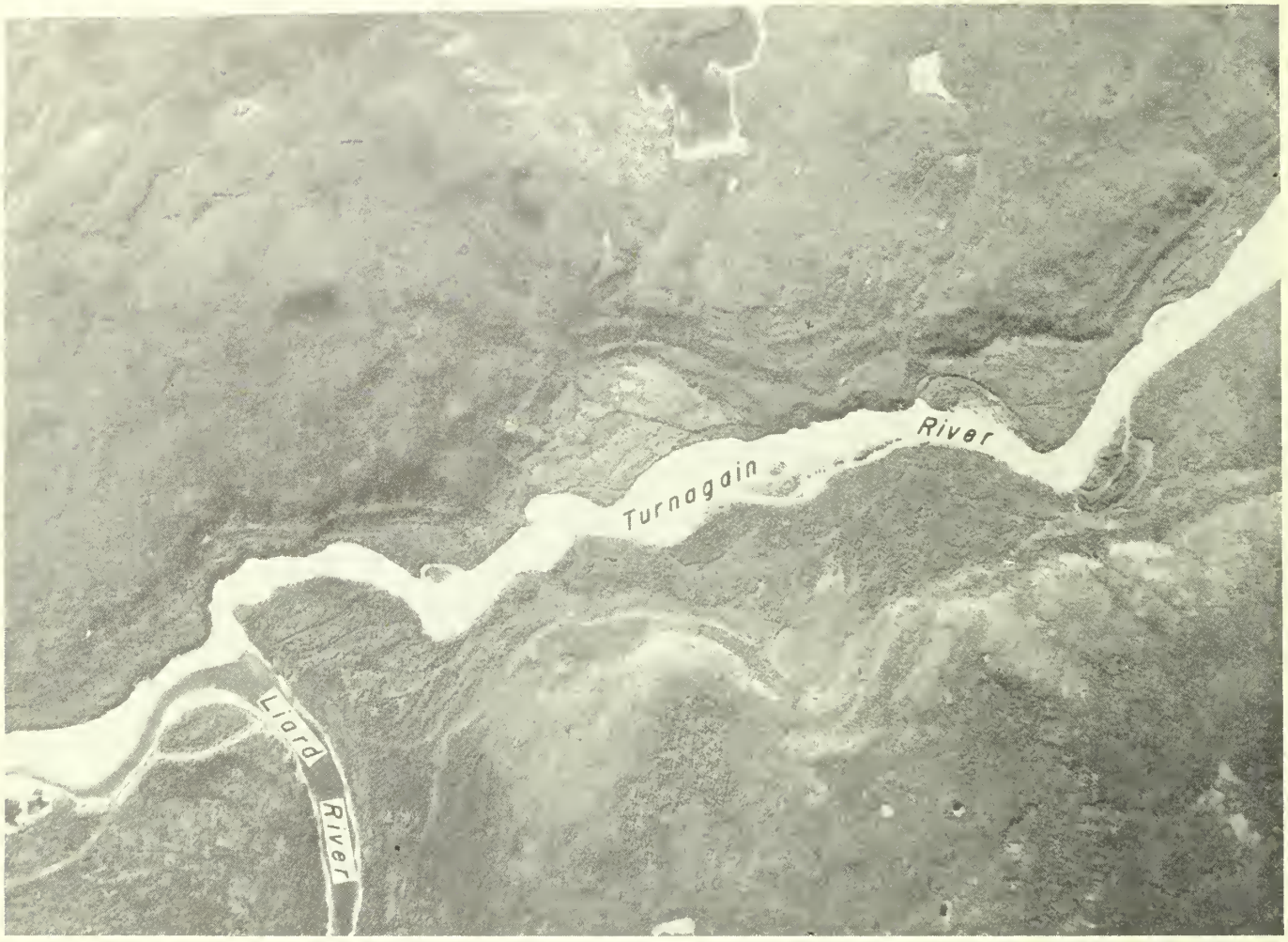

FIGURE 10.—A SMALL-SCALE VERTICAL PHOTOGRAPH; SCALE 2 INCHES PER MILE.

overlapping flights is shown in figure 13. These pictures show the area at the confluence of the Tetsa and Muskwa Rivers.

Controlled mosaics to accurate scale could not be made on the Alaska Highway. The shortness of time and the great difficulty of making ground surveys in the wild and inaccessible area made it impossible to establish the precisely located ground controls necessary for the assembly of controlled mosaics.

On the Alaska Highway the uncontrolled mosaics were usually assembled by the location engineers in their field camps. With a mosaic in hand, the engineer could locate his ground position and orient himself by compass bearings on prominent landmarks identifiable in the photographs. Distances could be scaled only approximately but planimetric accuracy at this stage of the location work was less important than finding suitable crossings of rivers and mountain passes, choosing between ridge and valley routes, locating passages across or around muskeg, and identifying soil and drainage conditions.

The aerial photographs for use in location of the Alaska Highway were taken with a 6-inch focal length lens camera, but experience gained by the War Department during World War II indicates that photographs taken with a 24-inch lens have a number of advantages for this type of work. With the longer focus lens photographic flights at an altitude of 14,400 feet will provide photographs at a scale of 600 feet per inch, permitting direct measurement of distances with an engineer's "60" scale. Scaling is not precise but is more nearly accurate than on the photographs taken with a 6-inch lens because the displacement of images due both to tilt of the plane and to variation in ground relief is reduced by the higher altitude of flight and the longer focal length of lens. The photographs cover an area about a mile wide on the standard 9-×9-inch negative and all essential ground details are clearly distinguishable. The 14,400-foot altitude is advantageous for flying since it is above the roughest air zones yet is not so high as to require the use of oxygen masks.

STEREOSCOPE PROVIDES PERCEPTION OF DEPTH

A photograph or mosaic, being a plane surface, has only two dimensions. Any simulation of a third dimension, which may occur to varying degrees, results from the perspective features of the picture and the subconscious interpretation of recognizable indicators of depth such as shadows.

An actual object or area is seen by the two eyes from slightly different angles, and this converging dual sight results in stereoscopic vision—the full perception of depth. If two photographs of an object taken from different camera positions are examined

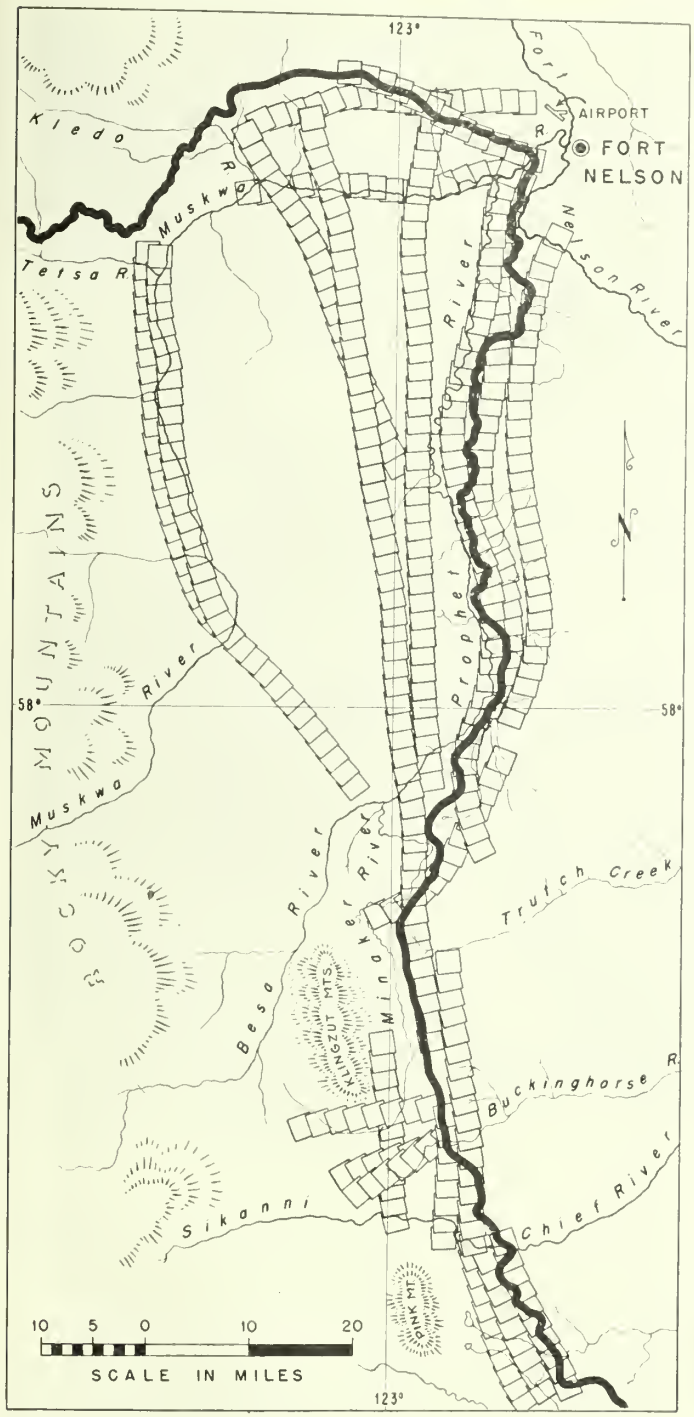

FIGURE 11.—LOCATION OF SOME OF THE FLIGHT LINES AND VERTICAL PHOTOGRAPHS TAKEN ALONG THEM ON ONE SECTION OF THE ALASKA HIGHWAY.

through a stereoscope, the effect of two-eye vision of the object is produced. Under the stereoscope the photographs seem to fuse together and take on the appearance of an actual three-dimensional model.

The topographic relief of an area may be studied in detail by the stereoscopic examination of aerial vertical photographs, and in this way the fullest use is made of the photographs for highway location work. Complete appreciation of this can be gained only by actually using a stereoscope.

FIGURE 12.—AN UNCONTROLLED MOSAIC MADE BY MATCHING A SERIES OF VERTICAL PHOTOGRAPHS.

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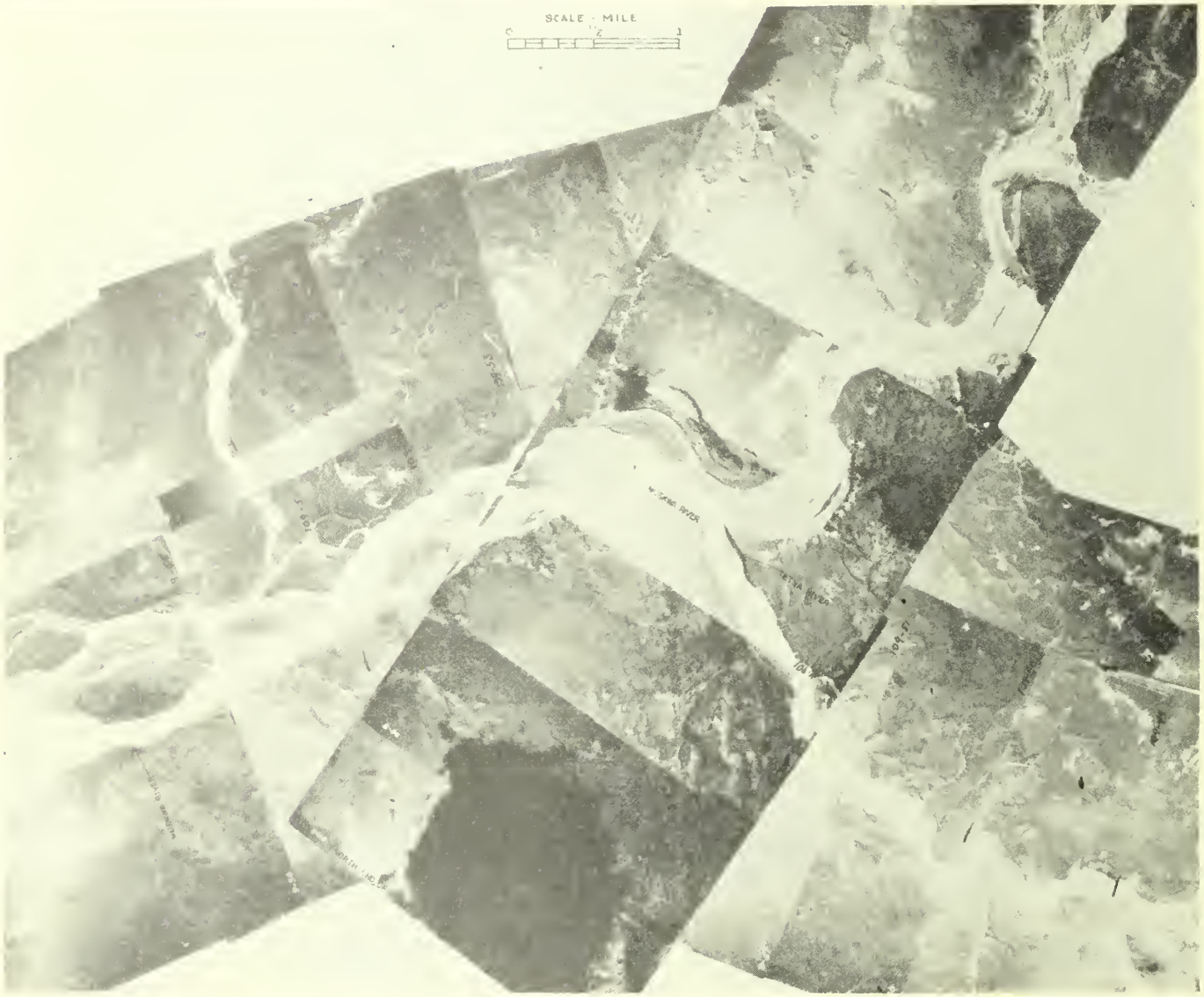


FIGURE 13.—AN UNCONTROLLED MOSAIC MADE OF PHOTOGRAPHS FROM SEVERAL OVERLAPPING FLIGHTS.

The parlor stereoscope, familiar to everyone a generation ago, is an application of stereoscopic observation developed largely for instructive entertainment. Several types of instruments, such as the simple lens stereoscope shown in figure 14 set up over a pair of photographs, are now available for engineers. When properly oriented, spaced, and mounted the pair of photographs, covering the same area but taken from different camera positions, is called a stereogram.

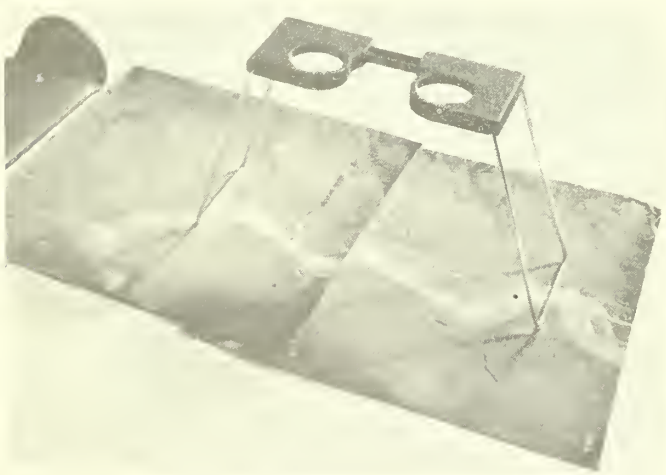


FIGURE 14.—AN ENGINEER'S STEREOSCOPE—AN AID IN SEEING PICTURED TOPOGRAPHY IN RELIEF.

A typical stereogram is shown in figure 15. The two larger circles represent the centers of the photographs—it was from above these points that the pictures were taken. The smaller circle drawn on the left photograph marks the transferred center of the right photograph. A mountainous area, much of it above the timber line, is shown in this stereogram. Though an idea of the direction of slopes can be guessed from the shadows, no real judgment of relative height or depth can be made until the three-dimensional image is seen under the stereoscope.

Some of the field engineers working on the Alaska Highway developed an ability to examine pairs of vertical photographs stereoscopically without the aid of an optical instrument. They could thus do their work more expeditiously and did not have to carry the lens stereoscope.

This ability to see stereoscopically without optical aids can be acquired by practice. At first, a cardboard

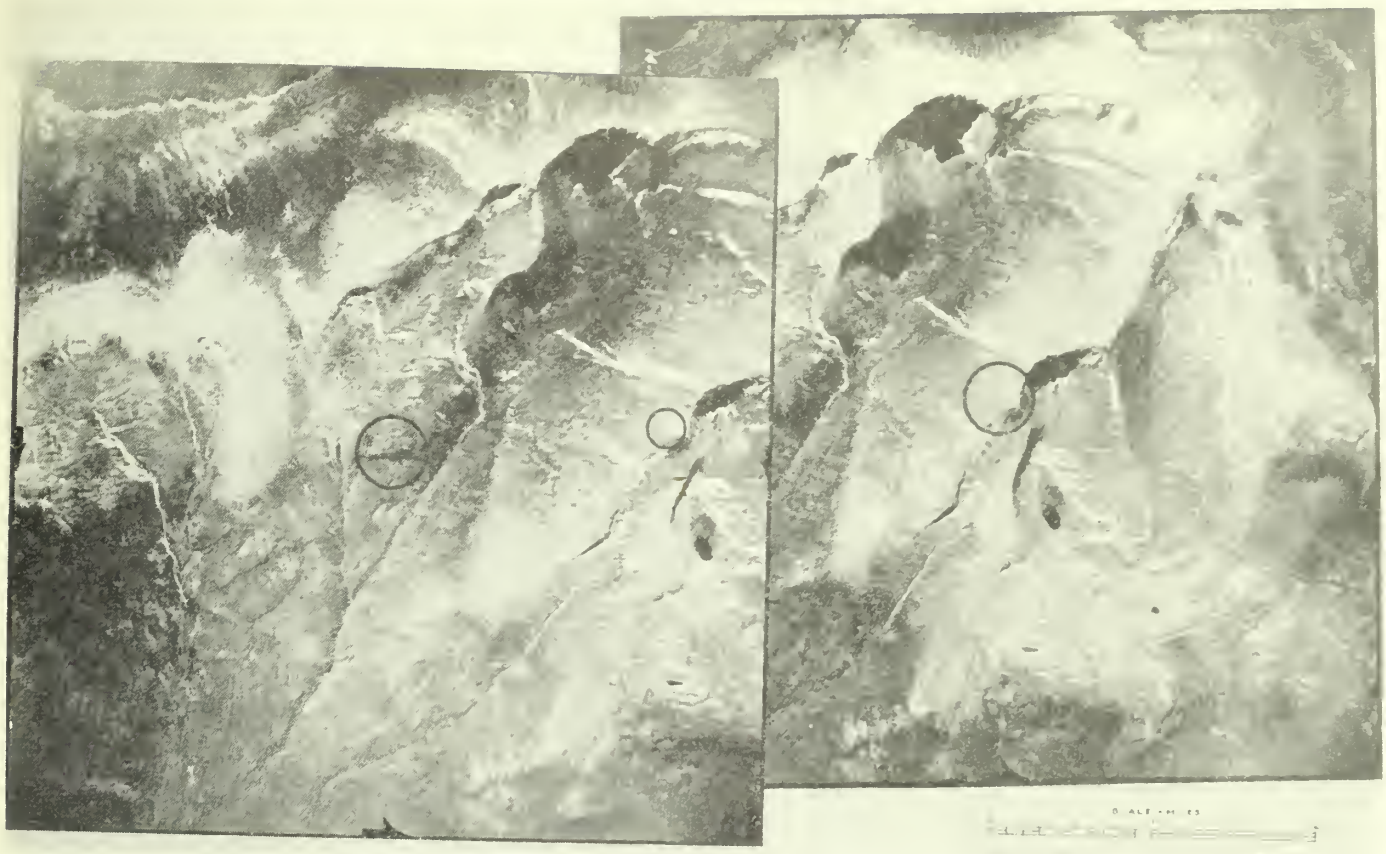


FIGURE 15.—A TYPICAL STEREOGRAM—A PAIR OF PHOTOGRAPHS ORIENTED FOR STEREOSCOPIC STUDY.

10 or 12 inches high should be placed on edge along the dividing line between the two photographs of the stereogram. A well-defined feature should be selected as a focal point: For example, the small lake in figure 15 which appears below the circled center in the right photograph and the circled transferred center in the left photograph. The forehead is rested on the top of the cardboard, with each eye looking at the selected point on the photograph beneath it. Both eyes must be focused as though looking beyond the plane of the stereogram until the chosen points appear to fuse together. The entire area within view will then be seen in three dimensions.

Some adjustment in the spacing and orientation of the two photographs and in the height of the eyes above them may be necessary. After some practice the cardboard may be dispensed with. Photographs are best studied stereoscopically under uniform lighting. They should be oriented so that the shadows in them incline toward the observer.

SOIL AND DRAINAGE CONDITIONS INTERPRETED

Photographs appear in realistic detail when seen stereoscopically. The valleys become depressions, peaks and ridges rise in full height, trees reach skyward, and all topographic features fit into their proper elevational position. The location engineers can see where routes might feasibly be placed; where obstacles would bar the way.

An additional and equally important purpose served by stereoscopic study of aerial photographs is the recognition of soils and drainage conditions from their relation to vegetation and topography, as shown in table 1. The engineers on the Alaska Highway had little knowledge of these relations at the beginning of the work and, in fact, none had been established for the subarctic region. These relations could be determined only by examination of the aerial photographs at identifiable places on the ground where the drainage conditions could be directly observed and the soils sampled and tested.

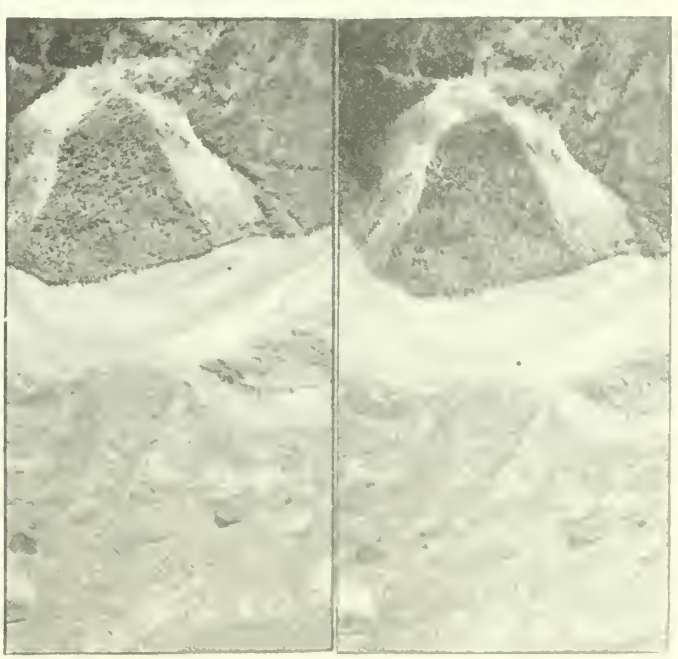


FIGURE 16.—A STEREOGRAM OF A RIVER VALLEY IN RUGGED TOPOGRAPHY.

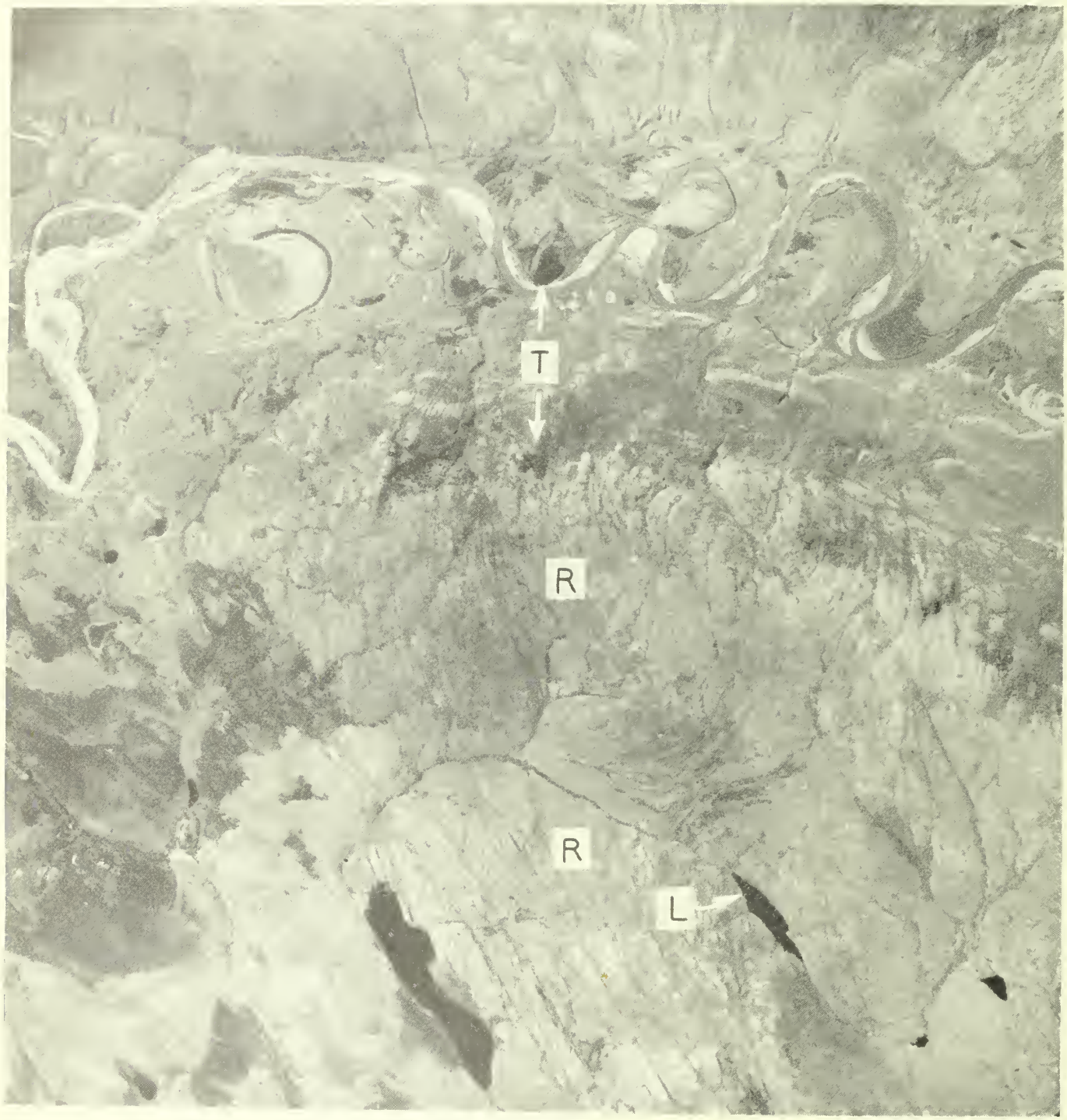


FIGURE 17.—GLACIATED, RUGGED TOPOGRAPHY. MUCH OF THE BEDROCK IS EXPOSED.

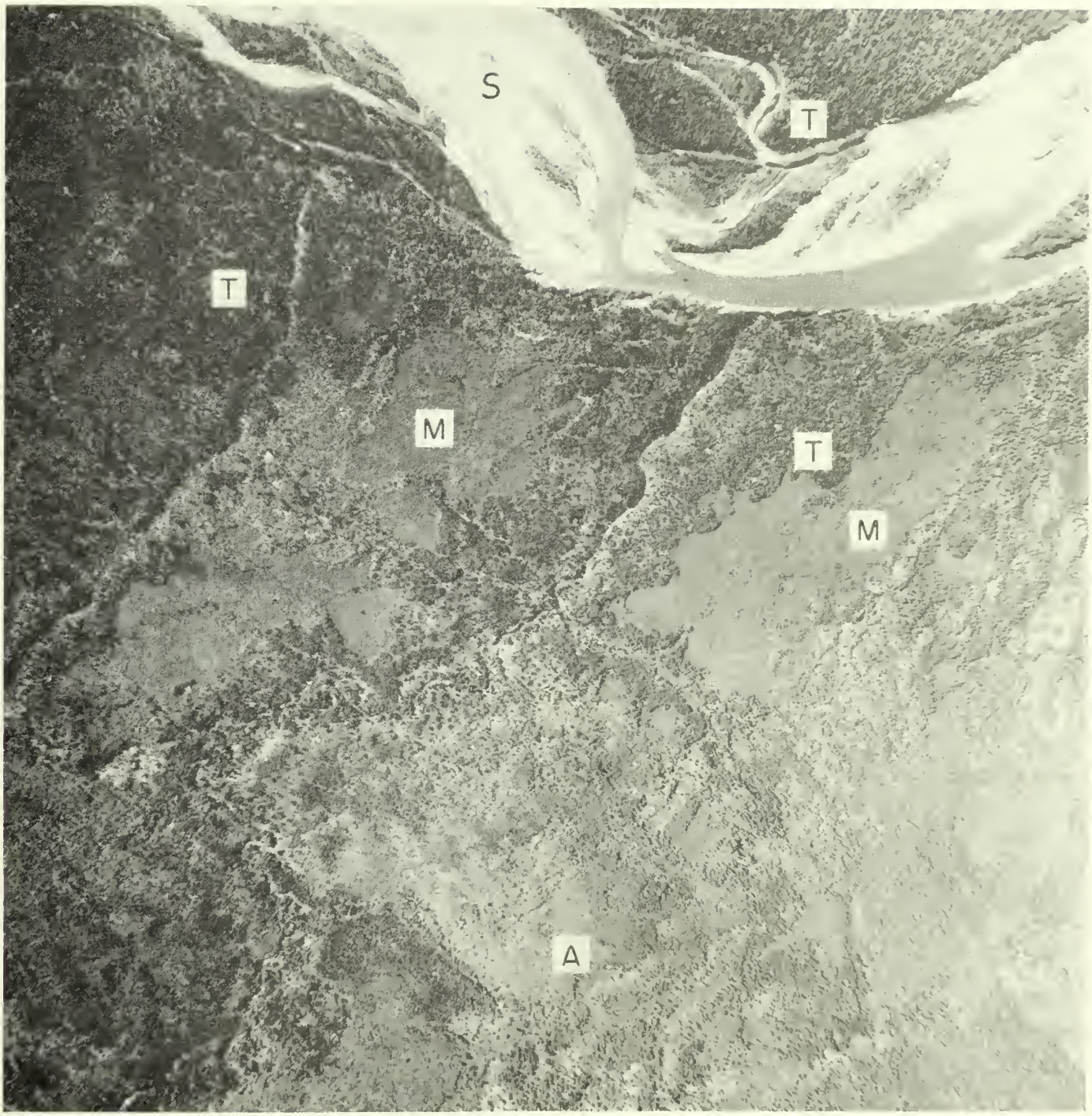


FIGURE 18.—A LEVEL PLATEAU CUT ACROSS BY A RIVER VALLEY.

TABLE 1.—*Vegetation as an indicator of soils and drainage* [1]

Type of vegetation	Soil	Drainage
Scrub spruce, willows, dwarf birch, bushes, moss.	Muskeg (sphagnum swamp), acid soil, impermeable base materials.[2]	Poor drainage or standing water.
Willows, birch, alder.	Sandy, silty, or sandy and gravelly.	Poorly drained or stream watered.
Balsam, cottonwood, white spruce.	River and stream deposits of silt, sand, and gravel.	High water table, wet sloughs between ridges of ground.
Evergreens (mostly white spruce) of large size for the locality.	Firm soils, mostly impermeable, on uneven but not rugged relief.	Drainage rather poor. If well watered, the soil may be granular and drainable.
Quaking aspen, in low relief areas.	Clay and silt, or lacustrine clays.	Muddy if worked. Water table is close to ground surface.
Quaking aspen, in rolling relief and on south slopes.	Permeable soils, gravelly sand and silts or clay.	Drainable but often damp or wet.
Jack pine	Sandy and gravelly, relief is usually uneven.	Dry or drainable.
Mixed evergreens of spruce, fir, and pine on glacial deposit areas.	Granular soils of gravel, sand and silt.	Permeable and drainable.

[1] The indications given are based on a study of average topographic conditions, and are subject to local variations. Aspen and jack pine tend to come in on all burned over areas in stage reforestation. Jack pine tends toward the more dry and granular soils, and aspen to south exposures. On very wet areas both types are soon replaced by other kinds of growth.

[2] Impermeable base in British Columbia may be fine clay and silt; in Alaska and Yukon it is usually permanently frozen ground.

Gradually experience was accumulated and the location engineers learned to interpret the nature of soils and drainage from the types of vegetation and topography seen stereoscopically in the vertical photographs. Some of these interpretations are illustrated in the examples that follow.

The stereogram, figure 16, shows a river valley in rugged topography. On the far side of the river, tall evergreens are growing on a slide area. The slide indicates unstable ground. On the near side, the steep banks sloping down to the river are covered with aspen and small evergreens. The small lakes lying on the shelf halfway up the slope are a sign of stable soil.

The vertical photograph shown in figure 17 covers an area roughly gouged by the ice-age glacier. A large part of the area pictured is exposed solid rock, *R*. The clear-water lake, *L*, appears dark because of aquatic plant growth which absorbs light. Scattered evergreen groves, *T*, grow in deposited soil, well watered but not undrainable, and usually of sandy or gravelly nature. The tallest timber (mostly spruce) is at the river bends and ox bows and indicates granular, watered soil. The meandering character of the river signifies a nearly level flow line. Between the river and the lakes there is a high rock divide.

The vertical photograph, figure 18, shows a level plateau above a river valley. White areas in the river bed, *S*, are sand, silt, and gravel, flooded during high water but dry when the picture was taken. The tallest evergreens, *T*, grow along the river where there is well watered but drainable granular soil. Aspen groves, *A*, with some scattered evergreens signify good drainage though the soil may not be granular. A ground view of this type of vegetation on rolling topography is shown in figure 19, A. Here the soils are likely to be permeable and drainable and usable for road building. Aspen often appears as a stage of natural reforestation after a forest fire.

The light-colored areas, *M*, in figure 18 are muskeg. The few trees growing within these areas are small and scattered. Muskeg is a certain indication of poor drainage, a considerable covering of organic material, and an impervious underlying base. The sticky ooze exposed whenever the muskeg cover is cut can be seen in figure 19, B. The pioneer road construction crews avoided muskeg whenever they could.

The stereogram, figure 20, shows glaciated topography with scattered evergreen groves. The eskers and drumlins among the small lakes cause the appearance of "pockmarks" and "knob and kettles." Glacial deposits of this character are definite signs of materials suitable for road building. The pioneer road can be seen in this picture winding across the rough topography. The more direct alinement built subsequently can also be seen.

Topography of the type illustrated in figure 20 is shown as it appears from the ground in figure 21. The largest trees grow in the watered valleys such as that crossed by the high fill in the center of the picture.

PIONEER ROAD LINES SELECTED FROM PHOTOGRAPHS

When the general line of a portion of the highway was determined, perhaps along a river valley or following a ridge, the location engineer in the field used the vertical photographs of the area in which he was working to plan his advance from point to point. Assembled as an uncontrolled mosaic, the photographs could be used to select several possible routes between the points chosen as controls.

FIGURE 19.—(A) MIXED VEGETATION ON ROLLING TOPOGRAPHY INDICATES SOILS LIKELY TO BE GOOD ROAD MATERIALS. (B) MUSKEG MAKES ROAD CONSTRUCTION DIFFICULT.

Each trial line was then studied in detail and was adjusted to the topography and to soil and drainage conditions as seen and interpreted by stereoscopic examination of successive pairs of photographs. The final lines were then drawn with a wax crayon on each vertical as illustrated in figure 22, which shows a glaciated area along the Rancheria River. The line had already been located to point *A* by study of adjacent photographs. From this point there are several possibilities: Route *A*—*B*—*C* is direct but crosses irregular ground; route *A*—*D*—*E* is longer but lies on fairly smooth topography; an alternate *A*—*D*—*B*—*C* combines parts of the two routes. The choice of going through *E* or *C* would depend on study of adjoining photographs.

The tentative locations selected on the photographs were examined on the ground by the survey engineers and the best route, modified if necessary to fit actual ground conditions, was flagged and blazed. The shortest route between control points such as river crossings, mountain passes, and passages around or through muskeg was followed wherever possible. High ground usually proved to be the best choice for rapid pioneer road building on the Fort St. John division of the highway.

Sometimes alteration or abandonment of the first trial lines selected from the photographs became necessary because of unsuitable soil conditions not evident in photographic interpretation. If possible, alternate lines were then selected by further study of the uncontrolled mosaic and stereoscopic examination of the vertical photographs. In this way the field engineers, in their tent camps, could choose tentative alternate lines without delay.

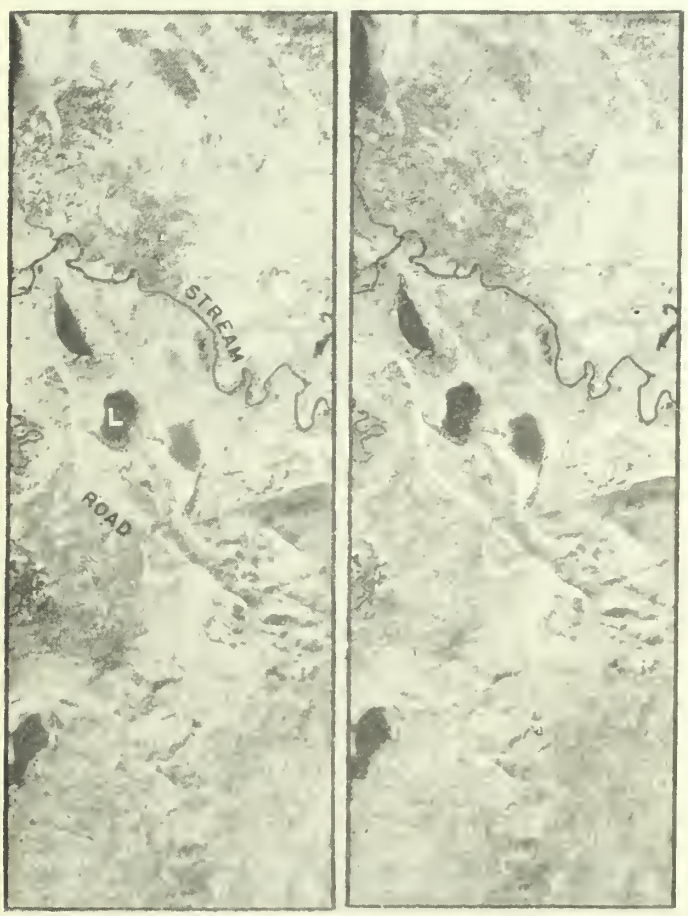


FIGURE 20.—GLACIAL DEPOSITS ARE EVIDENT IN THIS STEREOGRAM.

FIGURE 21.—GLACIAL DEPOSITS PROVIDE GOOD ROAD MATERIALS.

When a practicable route could not be found on the existing photographs, additional vertical photographs were requested along a new flight line. Often there was not time for this, however, as the construction crews frequently were close behind the survey parties. The location then had to be selected by ground reconnaissance which was usually difficult and occasionally resulted in overlooking a better route than the one actually selected.

FLAGGING AND BLAZING THE LINE

The line chosen by study of the photographs and tested on the ground was flagged and blazed for the bulldozer operators of the pioneer road construction crews to follow in the initial clearing operations. The location engineer used the mosaics and studied the photographs stereoscopically to keep himself oriented and the line aimed at a fixed objective. Where the topography was so rugged or irregular that acceptable grades could not be selected from the photographs with assurance, an Abney level was used.

If ground elevation control points had been established in advance, working grade lines could have been located on the paired photographs even in rugged topography with a simple elevation measuring instrument such as a parallax bar or a stereocomparagraph.

Occasionally the bulldozers would get off the blazed line. Sometimes at night they lost sight of the markers, and at other times they diverted from the line to avoid a small area of muskeg or unstable ground not noticeable until a tractor started to cross it.

If the machines were far off course and well ahead of the diversion point they had to be directed forward to the blazed line. In the haste to push the road through, backing up to start over again could not be permitted.

Direction of the bulldozers back to the blazed line was not unduly difficult if they were still within the area covered by the aerial photographs, as a course could be found by stereoscopic study of the paired verticals. When the machines had strayed beyond photographic coverage, however, the location engineer had to find a way back to the line as best he could by ground reconnaissance and compass directioning. This was always troublesome and time consuming.

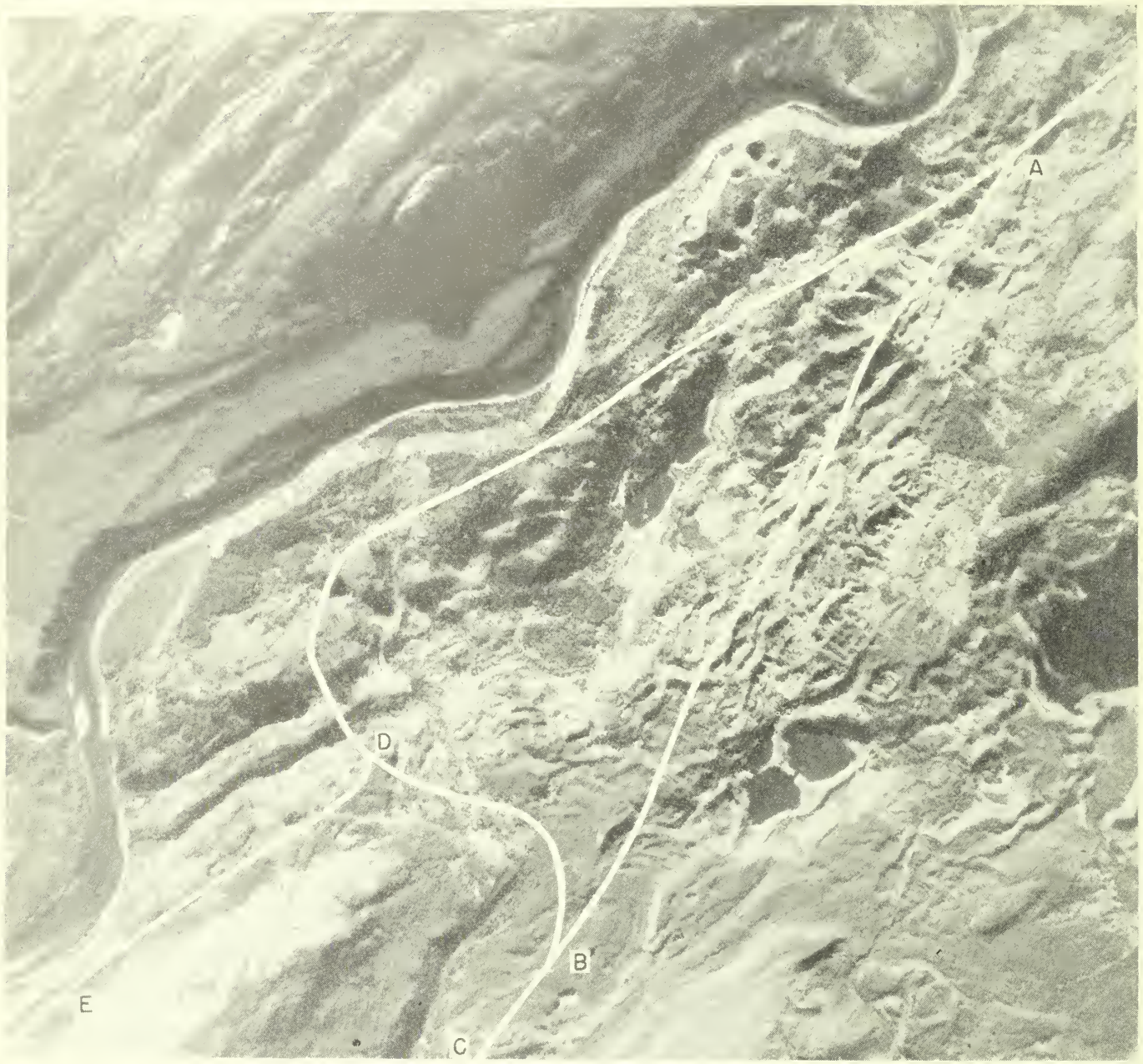


FIGURE 22.—TENTATIVE LOCATIONS SELECTED ON A PHOTOGRAPH.

ALASKA HIGHWAY LOCATION A PROVING GROUND FOR AERIAL SURVEYING

The engineers using aerial survey methods for location of the pioneer road of the Alaska Highway had in their hands a new technique. Their ability to apply it, and at times their confidence in it, were limited during the early days of the work. There was no alternative to adoption of the methods, however, for it would have been physically impossible, in the short time allotted, to locate the road entirely by ground survey through the wild, rough country that had to be traversed.

By trial and error the engineers acquired experience and eventually became adept in the use of the aerial photographs. Practical procedures were worked out, and principles of photographic interpretation were evolved for the subarctic region, though some of these principles were not established until the location work was well under way.

Many difficulties were encountered as a result of the lack of initial planning of the aerial survey work, for which inexperience, nonexistence of useful maps, and the driving need for haste were responsible. Some of the road had to be located by ground survey because aerial coverage could not be provided in time. When the photographs were available, however, their value was unquestionable, particularly after the field engineers had become proficient in their use. The Alaska Highway may well be considered a proving ground for highway location by aerial survey methods. The successful application of the methods there has helped to give impetus to their use.

RUBBER-ASPHALT JOINT FILLERS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by RICHARD H. LEWIS, Senior Chemist

Rubber-asphalt blends have been used for some years as joint fillers in concrete pavements and in general have been found superior to the more common types of hot-poured materials. The failure of some rubber-asphalt compounds to serve satisfactorily has been traced to overheating these materials and to holding them at high temperatures for long periods at the time of pouring. Such treatment causes chemical changes that alter the physical properties of the materials and consequently impair those characteristics which make the materials efficient joint fillers.

Laboratory tests indicate that rubber-asphalt compounds should be heated to closely controlled optimum temperatures predetermined for each particular blend. Batches should not be heated for extended periods, and an expeditious mechanical method of pouring should be employed.

Experiments show that the prolonged flow test required in the present specification for rubber-asphalt materials can be supplanted by the easier and quicker softening point test. The cone penetrometer test, also required at present, can be replaced by the standard needle penetration test.

FOR SOME YEARS joint-filling materials composed of blends of asphalt and rubber or rubbery synthetics have been used for sealing joints in concrete pavements. The service performances of joint fillers of this type, as indicated by inspection reports, show variable behavior. In some instances their performance has been superior to that of the more common types of hot-poured fillers, such as blown asphalt, refined asphalts, both petroleum and native, and mineral-filled asphalt. In other instances, their performance has been very unsatisfactory. Such variable behavior has been noted not only on the same projects but also on different projects with materials from the same source.

Producers have claimed, and present specifications indicate, that these materials may be heated satisfactorily for pouring without special precautions in the usual type of asphalt kettle, and that they can be poured satisfactorily at temperatures below 450° F. Laboratory studies and field experience do not substantiate these claims. Tests on materials from various sources have shown that some of these materials are difficult to liquefy and are of a ropey or nonuniform consistency unsuitable for pouring at the temperatures designated by the producer or provided for by existing specifications.

MATERIAL EXTENSIVELY USED FOR FILLING JOINTS IN CONCRETE RUNWAYS AND TAXIWAYS

Heating these materials in asphalt kettles in the field or over direct heat with gas burners or electric hot plates, which is common laboratory practice, often results in local overheating which causes an incipient decomposition of the rubber constituent of the blend. It has been found that even when the material is carefully heated in an oil-jacketed container precautions must be taken to control the softening process so that overheating does not occur.

In a report [1] by R. B. Jennings, which covers his inspection of installations of rubber-asphalt joint fillers in the eastern area of the United States, and more specifically the pouring of the material on the runways and taxiways at the Naval Air Test Center, Patuxent River, Md., the cause of the variable behavior of this material as a joint filler is attributed principally to damage of the material in heating. He claims that the use of the asphalt kettle, together with the retention of large quantities of the material at a high temperature for a long time, alters materially the physical properties of the filler causing the following types of failures:

(1) On crowned runways or taxiways or wherever grades were in excess of 1 percent the material exhibited a tendency to flow in joints during hot weather.

(2) The material lost resiliency and would not readily recover its original shape when deformed.

(3) The material had a surface stickiness; it would adhere to the tires of vehicles passing over it; and windblown sand and gravel striking the surface would adhere to it.

(4) Sand and small stone adhering to the surface of the joints settled into the jointing material by reason of their higher specific gravity and, in some cases, displaced the joint material leaving an incompressible material in the joint which would result in spalling of the concrete.

The major part of Jennings' report [1] deals with the development of equipment and methods for heating the joint filler to pouring consistency without subjecting the material to direct heat. Mechanical methods are also described which expedite the filling of the joint. The modifications in the preparation and handling of this material on more recent work have resulted in much improvement in the behavior of the installed joints.

FEDERAL SPECIFICATIONS PROVIDE FOR RUBBER-ASPHALT JOINT

This report deals primarily with the heating and pouring of rubber-asphalt joint-filling materials. Data obtained from a laboratory study of five commercial joint fillers from four sources are presented. Two of these materials, samples B and C, represent two different shipments from the same producer. All of these materials were submitted to meet Federal Specification SS–F–336, the test requirements and test methods for which are given below.

D. GENERAL REQUIREMENTS.

D–1. Material furnished under this specification shall be suitable for melting in the usual type of asphalt kettle. It shall be capable of melting readily and uniformly to a pouring consistency at a temperature of not more than 450° F. and shall comply with the detail requirements specified in section E. The material shall not be damaged when heated to the temperature required for satisfactory pouring.

[1] New Developments in Pouring Concrete Pavement Joint Filler, by Commander R. B. Jennings (C. E. C.) U. S. N. R., presented at the American Road Builders' Association annual convention, Chicago, Ill., January 1946. This report, somewhat modified, was printed as The Design and Maintenance of One and a Half Million Square Yards of Paving Slab to Prevent Pumping of Subgrade Through Paving Joints, in the American Road Builders' Association Technical Bulletin No. 103, 1946, pp. 14–25.

TABLE 1.—*Effect of pouring temperatures on test characteristics of rubber-asphalt joint fillers*

	Characteristics of material A at pouring temperature of—				Characteristics of material B at pouring temperature of—				Characteristics of material C at pouring temperature of—				Characteristics of material D at pouring temperature of—				Characteristics of material E at pouring temperature of—			
	400° F.[1]	450° F.	500° F.	550° F.	400° F.	450° F.[1]	500° F.	550° F.	400° F.	450° F.[1]	500° F.	550° F.	400° F.	450° F.[1]	500° F.	550° F.	400° F.	450° F.	500° F.[1]	550° F.
Time of heating to pouring temperature min	35	35	30	40	55	50	35	60	50	40	40	55	40	40	40	80	55	50	50	70
Penetration, needle, 100 gm., 5 sec.:																				
At 59° F	52	60	56	59	34	39	48	32	48	60	57	32	36	34	36	39	38	48	57	58
At 77° F	85	102	111	120	58	66	87	65	86	95	113	79	59	63	66	82	63	79	108	109
At 95° F	145	166	196	231	97	113	157	298	132	154	205	293	100	116	116	160	99	135	171	190
Slope of log-penetration-temperature curve	0.0124	0.0123	0.0151	0.0165	0.0126	0.0128	0.0133	0.0269	0.0122	0.0114	0.0154	0.0267	0.0123	0.0148	0.0141	0.0170	0.0116	0.0125	0.0133	0.0143
Penetration, cone, 150 gm., 5 sec.:																				
At 59° F	43	47	36	47	34	42	43	25	51	54	56	32	31	26	25	33	37	42	52	49
At 77° F	76	79	81	104	53	64	69	47	79	84	100	74	49	47	49	64	50	69	90	84
At 95° F	111	125	138	159	81	97	135	132	109	128	170	169	75	76	78	115	75	105	133	142
Softening point (ring and ball) ° F.	192	167.5	149.0	139.1	229.6	200.3	164.3	141.8	183.7	176.9	154.9	147.2	212.9	187.3	190.0	150.4	246.2	239.5	188.2	148.1
Ductility test at 41° F.:																				
Ductility, 5 cm./min cm	11.5	12.5	6.5	8.5	13.5	14.5	15.0	12.5	19.0	22.0	21.5	12.0	5.5	5.0	5.0	5.5	23.5	42.5	61.0	44.0
Length of broken specimen after 15 min cm	3.6	4.2	3.3	4.7	1.8	2.0	3.8	8.7	3.9	4.4	6.4	7.1	4.4	2.7	3.2	3.7	7.5	10.7	15.4	18.0
Rebound after 15 min cm	7.9	8.3	3.2	3.8	11.7	12.5	11.2	3.8	15.1	17.6	15.1	4.9	1.1	2.3	1.8	1.8	16.0	31.8	45.6	26.0
Percentage rebound percent	69	66	49	45	87	86	75	30	79	80	70	41	20	46	36	33	68	75	75	59
Ductility test at 77° F.:																				
Ductility, 5 cm. min cm	16.0	15.0	11.5	10.0	16.0	17.0	15.0	11.5	19.5	25.0	20.5	12.5	9.0	10.5	5.0	8.0	27.0	40.0	114.0	228
Length of broken specimen after 15 min cm	7.3	7.2	6.6	5.2	6.2	7.1	6.2	7.3	9.4	12.0	8.3	7.0	5.3	7.5	3.0	6.3	8.6	10.0	31.0	85
Rebound after 15 min.. cm	8.7	7.8	4.9	4.8	9.8	9.9	8.8	4.2	10.1	13.0	12.2	5.5	3.7	3.0	2.0	1.7	18.4	30.0	83.0	143
Percentage rebound percent	54	52	43	48	61	58	59	37	52	52	60	44	41	29	40	21	68	75	73	63
Flow at 140° F. in 5 hours cm.	0.3	1.2	18.3	15.4	0	0.1	1.4	20.6	0.3	0.6	2.4	15.4	0	0.2	0.4	14.3	0	0.1	0.4	10.1

[1] Minimum suitable pouring temperature for this material.

E. DETAIL REQUIREMENTS.

E–1. Material furnished under this specification when tested in accordance with the methods described in section F shall conform to the following requirements:

(*a*) Pour point, not more than ..... 450° F.
(*b*) Melting time, not more than ..... 60 min.
(*c*) Penetration:
At 32° F., 200 g., 60 sec., not less than.. 0.28 cm.
At 77° F., 150 g., 5 sec..... 0.45 to 0.75 cm.
(*d*) Flash point, not less than..... 550° F.
(*e*) Flow, not more than..... 0.5 cm.
(*f*) Bond test.
There shall be no cracking of the material or failure in bond between the materials and the mortar test blocks at the end of five cycles.

F. METHODS OF TESTING.

F–1. The following methods of testing shall be used in determining compliance with the detail requirements specified in section E:

F–1a. *Preparation of sample.*

F–1a (1). A sample of the material weighing approximately 300 grams shall be selected in such a manner as to avoid inclusion of the surface layer. Of this quantity, 100 grams shall be melted, with continued stirring to a pouring consistency, in a clean container, using an oil bath or similar heating unit. The operation of melting shall be conducted as rapidly as possible without danger of local overheating. The remaining 200 grams shall then be added, in quantities of approximately 50 grams at a time, to the melted material until the entire quantity is of a sufficiently fluid consistency to be poured satisfactorily.

F–1b. *Pour point.*

F–1b (1). The minimum temperature at which the material will pour readily and uniformly when subjected to the procedure described in section F 1a (1).

F–1c. *Melting time.*

F–1c (1). The time in minutes required to melt the material to pouring consistency when subjected to the procedure described in section F 1a (1).

F–1d. *Penetration.*

F–1d (1). The penetration test shall be made in accordance with the procedure described in Federal specification SS–R–406, for the penetration test of bituminous materials, except that a penetration cone conforming to the following requirements shall be used in place of the standard penetration needle: The cone shall be constructed of stainless steel or brass with a detachable hardened-steel or stainless steel tip. It shall conform to the dimensions shown in figure 1 of A. S. T. M. Method of Test D 217–38T except that the interior construction may be modified as desired. The outside surface of the cone and tip shall be given a very smooth finish. The total moving weight of the cone and attachments shall be 150 g.

F–1e. *Flash point.*

F–1e (1). The test for flash point shall be made in accordance with the procedure described in Federal specification SS–R–406 for the flash and fire point test by means of open cup.

F–1f. *Flow test.*

F–1f (1). A portion of the sample prepared as described in Section F–1a (1) shall be poured into a suitable amalgamated mold 4 cm. wide by 6 cm. long, placed on a bright tin panel. The sample shall be poured to a uniform depth of 0.32 cm. After cooling at room temperature for 2 hours, the mold shall be removed and the panel containing the sample shall be placed in an oven maintained at 140° F. ± 2° F. for 5 hours. During the test the panel shall be mounted at an angle of 75 degrees with the horizontal. The total movement in centimeters of the specimen during the 5-hour test period shall be reported as the flow.

F–1g. *Bond test.*

F–1g (1). *Extension machine.*—The extension machine used in the bond test shall be so designed that the specimen can be expanded at a uniform rate of approximately 0.125 inch per hour for at least 4 hours. It shall consist essentially of one or more screws rotated by an electric motor through suitable gear reductions. Self-alining plates or grips, one fixed and the other carried by the rotating screw or screws, shall be provided for holding the test specimen in position during the test.

F–1g (2). *Mortar blocks.*—Two cement mortar blocks each 1 by 2 by 3 inches in size shall be prepared, using one part of portland cement to two parts by weight of clean, uniformly graded concrete sand, all of which passes a No. 4 sieve and not more than 5 percent of which passes a No. 100 sieve. Sufficient water shall be used to produce a flow of 100 ± 5 when tested in accordance with the procedure described in Federal Specification SS–C–158a, section F–4m (4). After curing 1 day in moist air and 6 days in water at 70° F., the 2- by 3-inch faces of each block shall be surfaced by grinding with a No. 30 HD carborundum stone until the aggregate is uniformly exposed. The blocks shall then be air dried at room temperature for 24 hours prior to use.

F–1g (3). *Test specimen.*—The mortar blocks, prepared as described in section F–1g (2), shall be placed on a plate of amalgamated metal and shall be spaced 1 inch apart by means of amalgamated metal strips placed at such distances from the ends that an opening 1 by 2 by 2 inches is formed. The test specimen is completed by filling this opening with the material prepared in accordance with the procedure described in section F–1a (1), followed by cooling in air at room temperature for 2 hours. The mortar test blocks may be held together by means of clamps, rubber bands, or other suitable means.

F–1g (4). *Extension at low temperature.*—The test specimen, prepared as described in section F–1g (3), shall be placed in an atmosphere maintained at 0° F. ±2° F. for not less than 6 hours, after which the amalgamated plates and strip shall be removed and the specimen mounted immediately in the self-alining clamps of the extension machine described in paragraph F–1g

TABLE 2.—*Effect of time of heating at pouring temperature on test characteristics of rubber-asphalt joint fillers*

	Effect of heating material A at 400° F.[1] for—				Effect of heating material C at 450° F.[1] for—				Effect of heating material D at 450° F.[1] for—			Effect of heating material E at 500° F.[1] for—				
	0 hours	2 hours	3 hours	4 hours	0 hours	2 hours	3 hours	4 hours	0 hours	4 hours	5 hours	0 hours	1 hour	4 hours	5 hours	7 hours
Time of heating to pouring temperature—min.	35	40	65	45	40	30	40	35	40	45	40	50	40	50	35	40
Penetration, needle, 100 gm., 5 sec.:																
At 59° F.	52	45	51	50	60	58	56	31	34	35	34	57	68	33	29	26
At 77° F.	85	74	97	91	95	82	101	62	63	66	65	108	95	64	59	48
At 95° F.	145	139	155	157	154	183	202	128	116	101	118	171	141	97	97	68
Slope of log-penetration-temperature curve	0.0124	0.0136	0.0134	0.0138	0.0114	0.0139	0.0155	0.0171	0.0148	0.0129	0.0150	0.0133	0.0088	0.0130	0.0146	0.0116
Penetration, cone, 150 gm., 5 sec.:																
At 59° F.	43	39	47	41	54	52	46	28	26	26	27	52	58	30	26	23
At 77° F.	76	69	82	82	84	81	74	48	47	48	49	90	85	55	52	39
At 95° F.	111	116	132	132	128	143	162	112	76	78	87	133	126	86	85	59
Softening point (ring and ball)—°F.	192.2	159.4	165.2	155.3	176.9	152.6	149.5	147.7	187.3	167.5	168.8	188.2	163.9	180.5	176.5	190.4
Ductility test at 41° F.:																
Ductility, 5 cm./min—cm.	11.5	10.5	13.5	14.0	22.0	18.5	11.5	10.0	5.0	4.5	4.5	61.0	35.0	8.5	7.0	7.0
Length of broken specimen after 15 min.—cm.	3.6	3.2	5.3	5.8	4.4	6.5	5.0	4.7	2.7	3.2	2.7	15.4	8.3	2.7	2.3	1.8
Rebound after 15 min.—cm.	7.9	7.3	8.2	8.2	17.6	12.0	6.5	5.3	2.3	1.3	1.8	45.6	26.7	15.8	4.7	5.2
Percentage rebound—percent	69	70	61	59	80	65	57	53	46	29	40	75	76	68	67	74
Ductility test at 77° F.:																
Ductility, 5 cm./min—cm.	16.0	14.5	19.0	21.5	25.0	19.5	13.0	15.0	10.5	6.0	6.0	114.0	83.5	11.5	10.5	9.0
Length of broken specimen after 15 min.—cm.	7.3	7.0	8.0	8.9	12.0	9.0	5.8	8.5	7.5	3.8	3.8	31.0	24.0	2.4	2.6	2.2
Rebound after 15 min.—cm.	8.7	7.5	11.0	12.6	13.0	10.5	7.2	6.5	3.0	2.2	2.2	83.0	59.5	9.1	7.9	6.8
Percentage rebound—percent	54	52	58	59	52	54	55	43	29	37	37	73	71	79	75	76
Flow at 140° F. in 5 hours—cm.	0.3	2.7	1.2	5.3	0.6	5.6	8.5	15.7	0.2	1.4	2.7	0.4	3.0	0.5	1.0	0.3

[1] Minimum suitable pouring temperature for this material.

(1). The specimen shall then be extended at a uniform rate of 0.125 inch per hour for 4 hours. During this period the atmosphere surrounding the test specimen shall be maintained at a temperature of 0° F. ±2° F.

F–1g (5). *Recompression.*—After extension as described in section F–1g (4), the specimen shall be removed from the extension machine and allowed to remain in air at room temperature for 2 hours, after which it shall be compressed to its original thickness at the rate of approximately 0.1 inch per minute. The specimen shall be maintained in a horizontal position during compression.

F–1g (6). *Number of cycles.*—Five cycles of extension at low temperature followed by recompression shall constitute one complete test for bond as specified in section E–1 (f).

TESTING PROCEDURE USED IN THIS STUDY

The effect of the pouring temperature on the test characteristics was determined in the first series of laboratory tests. In this series the materials were heated, according to method F—1a(1) of Specification SS—F—336, to the temperatures shown in table 1, and then poured immediately into the testing forms. In liquefying these materials, the oil bath was brought to a temperature 50° F. above the selected pouring temperature and controlled within ±5° F. thereafter until the filler material reached the designated pouring temperature. When 550° F. was used as a pouring temperature an air bath was substituted for the oil bath when material became fluid, to accelerate the heating.

The specification describes the pour point as the minimum temperature at which the material will pour readily and uniformly. The literature of one producer states that the material is considered at its pour point when it has a uniform consistency and is capable of being poured into a ½-inch joint. The temperatures at which the materials were considered as having a suitable consistency for pouring are given in table 1.

The minimum suitable pouring temperatures as determined in these tests were above the temperatures recommended by the producers and, with the exception of sample A, were equal to or above the specification requirement.

The effect of maintaining the fillers at the minimum suitable pouring temperature for various periods of time is shown in table 2. Because of insufficient material sample B was not included in this phase of the study. In preparing these samples for test, the oil bath was heated as previously described. As the materials reached the designated pouring temperature, the bath temperature was so adjusted as to keep the temperature of the filler material within ±5° F. of the pour point during the stipulated heating period.

The laboratory tests made on the filler materials and the test procedures used are those required by Federal Specification SS–F–336 with the following exceptions:

Flash point determinations were not made because of the tendency of these materials to foam and overflow the flash cup while heating and also because of the difficulty of maintaining a constant rate of increase in temperature.

All of these fillers passed the bond test when tested in the manner prescribed in the specifications. Because of insufficient material, however, the bond test was not made on materials subjected to the varying conditions of this study.

The following tests not required by the specifications were made also to show the behavior of these joint fillers under variable laboratory heating and pouring conditions.

Penetration tests using the standard penetration needle under a load of 100 grams for 5 seconds were made at three temperatures, 59°, 77°, and 95° F.

Cone penetration tests were not made at 32° F., 200 grams, 60 seconds. These were made, however, at 59°, 77°, and 95° F. with a load of 150 grams for 5 seconds.

Softening points were determined by the ring-and-ball method using glycerin instead of water for temperature control.

Ductility tests, using standard equipment, were run at 41° and 77° F. at a speed of 5 centimeters a minute. After breaking, the test specimens were left undisturbed in the ductility bath for 15 minutes, after which the total amount of contraction or rebound in the broken halves of the elongated specimens was determined. The amount of rebound has been suggested by one producer as a measure of the resiliency of the material.

The cone penetrometer, which is required by Federal Specification SS–F–336 for the determination of the consistency of these asphalt-rubber blends, was devised initially for the control of greases and petrolatum. It

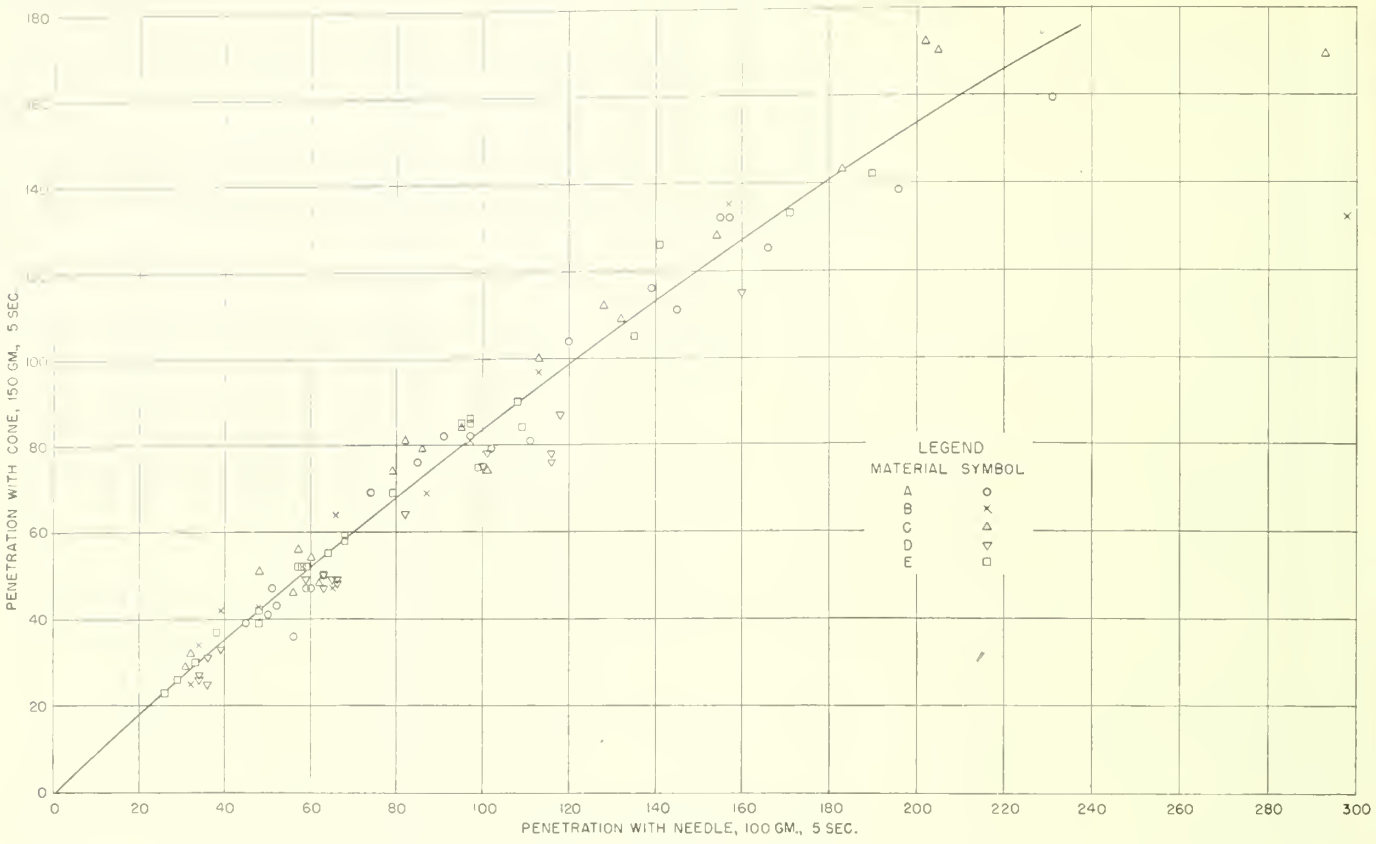


FIGURE 1.—RELATION BETWEEN PENETRATION WITH GREASE CONE AND STANDARD NEEDLE.

has been used, however, for determining the consistency of calking materials and other materials of nonuniform texture. The first rubber-asphalt joint-filling materials contained vermiculite or other flaky matter which to a certain extent prevented close checking of consistency with the standard asphalt penetration needle. It was found in this study that if the same number of tests are made with the standard penetration needle as are required by the A. S. T. M. method for the cone penetrometer, an average of the results is a satisfactory measure of the consistency of the filler material.

The results of the penetration tests, using the cone and needle penetrometers at three temperatures, are plotted in figure 1. Although there are a few points of variance the relation between the results of the two tests is very good. Since the standard penetration test is more familiar to highway engineers, the results obtained with the cone penetrometer will not be discussed further.

PENETRATIONS AT NORMAL TEMPERATURE ALTERED DURING HEATING

The data for the standard asphalt penetration tests given in tables 1 and 2 are plotted in figures 2 and 3. In figure 2 it will be noticed that as the pouring temperature increases, the penetration values at 95° F. increase, the increase being greater for samples B, C, and A than for samples D and E. The penetration values at 77° F. increase in like order but for samples B and C decrease for the material heated to 550° F. The effect of the pouring temperature on the penetrations at 59° F. is not as noticeable as for the other two temperatures. At this temperature no appreciable difference in the penetration values of the materials heated to various pour temperatures is obtained on samples A and D. In the case of samples B and C, although the penetrations at 95° F. of the materials heated to 550° F. greatly increase, the penetrations at 59° F. decrease. With sample E the penetration at 59° F. increases with the increase of the pouring temperature.

In figure 3 the effect of time of heating at the minimum suitable pouring temperature on the penetration values is shown to be more pronounced for samples C and E than for samples A and D. Prolonged heating of sample E tends to reduce the penetration values at the three test temperatures. In the case of sample C, the penetrations at 95° F. after 2 and 3 hours of heating increase but after a 4-hour heating period the penetrations at the three temperatures are considerably lower than values obtained for any of the other heating periods. The effect of time of heating on the penetration values of samples A and D show no consistent trend. A comparison of the data plotted in figures 2 and 3 shows that holding the materials at the selected pouring temperatures does not alter the penetration values as much as the subjection of the materials to higher temperatures before pouring.

In tables 1 and 2 the slopes of the penetration (plotted logarithmically)-temperature curves based on the penetration values obtained with the standard needle are given. In most cases, if the values shown are plotted to scale, there is approximately a straight-line relation between the temperature of test and the logarithm of the corresponding penetration.

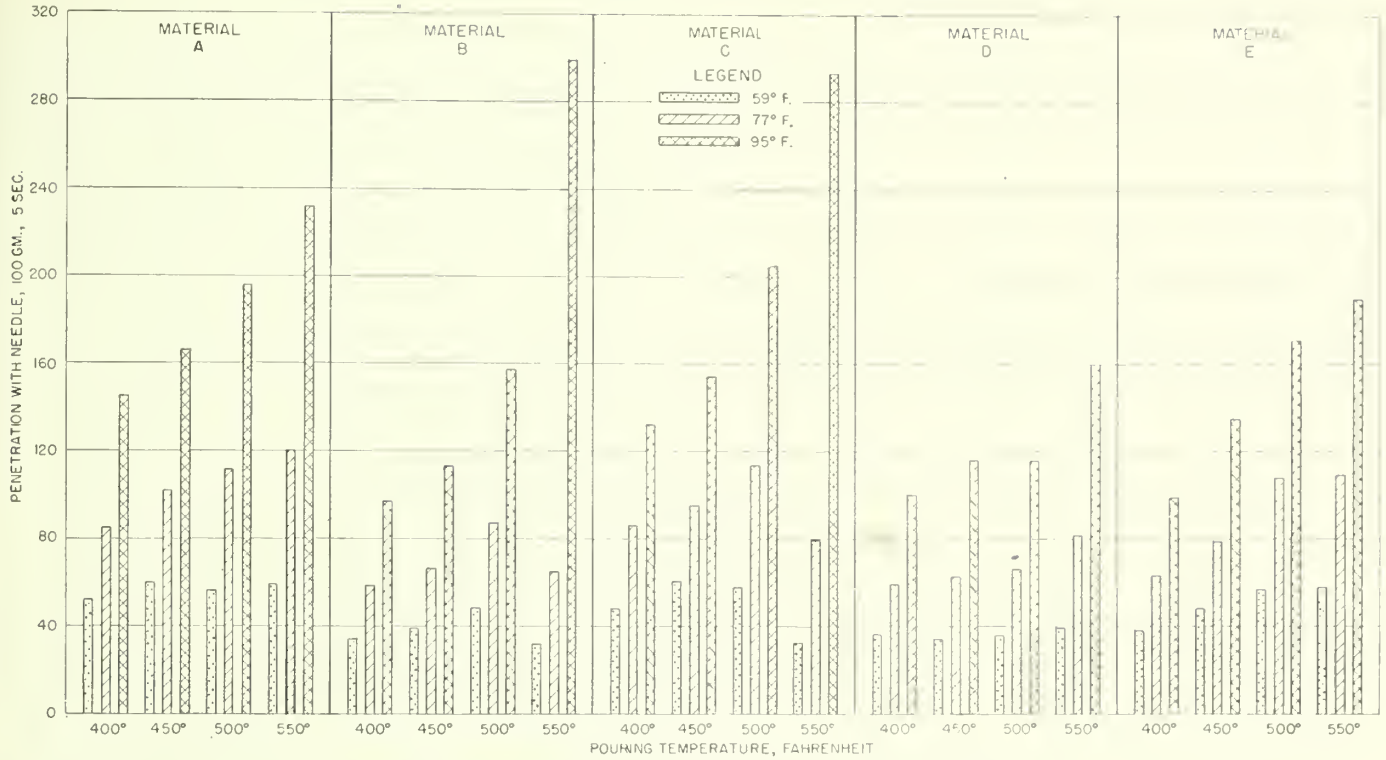


FIGURE 2.—EFFECT OF POURING TEMPERATURE ON PENETRATION.

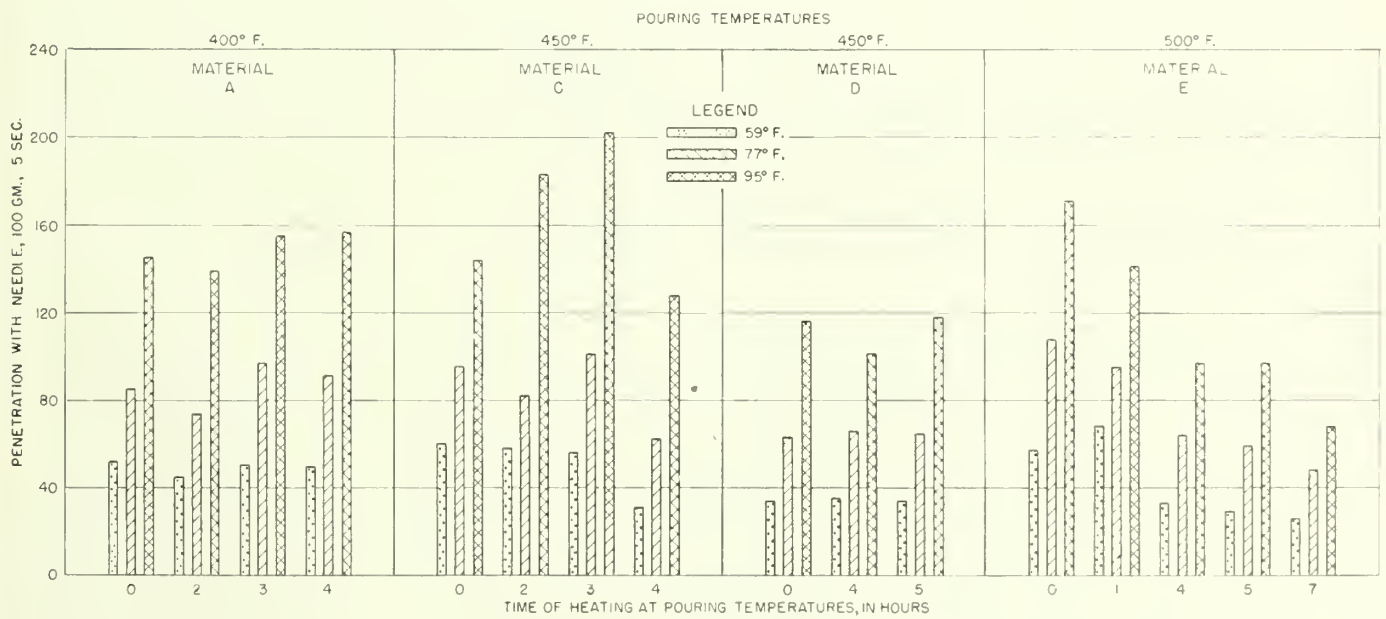


FIGURE 3.—EFFECT OF TIME OF HEATING AT MINIMUM SUITABLE POURING TEMPERATURE ON PENETRATION.

TEMPERATURE SUSCEPTIBILITY OF MATERIALS ALTERED

It has been established [2] that the log-penetration-temperature curves of 50–60 and 85–100 asphalts assume the form of straight lines, and a true indication of the susceptibility to change in temperature of their consistencies is found in the slope of the lines. The slope, as reported in tables 1 and 2, can be calculated as follows:

$$\text{Slope}=M=\frac{\log p_2-\log p_1}{t_2-t_1}$$

where p_2 and p_1 are the penetrations at the temperatures $t_2=95°$ F. and $t_1=59°$ F., respectively.

[2] The Physical and Chemical Properties of Petroleum Asphalts of the 50–60 and 85–100 Penetration Grades, by R. H. Lewis and J. Y. Welborn, PUBLIC ROADS vol 21, No. 1, March 1940.

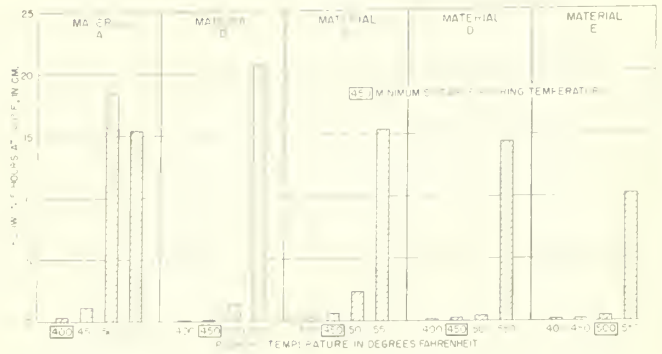


FIGURE 4.—EFFECT OF POURING TEMPERATURE ON THE FLOW TEST RESULTS.

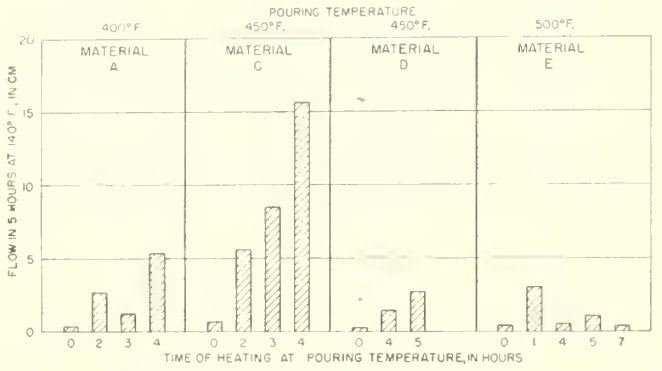


FIGURE 5.—EFFECT OF TIME OF HEATING AT MINIMUM SUITABLE POURING TEMPERATURE ON THE FLOW TEST RESULTS.

A study of these data as given in the two tables shows that as the pouring temperatures increase, the susceptibility of the poured rubber-asphalt blends to temperature change increases. The increase is highest for samples B and C and lowest for sample E. The effect of time of heating at pouring temperatures is generally progressive, is greatest for sample C, and is less pronounced and more variable for the other materials.

A series of blown asphalts commonly used as joint fillers in brick and concrete pavements have slopes of log-penetration-temperature curves ranging from 0.0090 to 0.0125, with an average of 0.0105. The log-penetration-temperature curves of 50–60 penetration asphalts from representative sources [2] have slopes ranging from 0.0172 to 0.0307 with an average of 0.0230, and for 85–100 penetration asphalts the slopes range from 0.0188 to 0.0324 with an average of 0.0242. It can be seen from the above values that the susceptibility of these rubber-asphalt blends initially is but slightly higher than that of blown asphalts; and that high temperatures tend to increase their susceptibility. The susceptibility of samples B and C when heated to 550° F. approximates that of the usual asphalt cement. Prolonged heating at the minimum suitable pouring temperature in most cases also increases the temperature susceptibility of these materials. It is apparent, therefore, that both the pouring temperature and time of heating must be carefully controlled to insure that these materials will have the proper consistency under all temperature conditions to perform satisfactorily as a joint filler.

[2] See footnote on p. 295.

SOFTENING POINT INDICATIVE OF FLOW PROPERTIES OF JOINT FILLERS

The effect of high summer temperatures on the fluidity of the asphalt-rubber filler is presumed to be indicated by the behavior of the material in the flow test. The results of the flow test, as given in tables 1 and 2, are shown graphically in figures 4 and 5. Table 1 shows that, with the exception of sample C, the flow of the materials poured at the minimum suitable pouring temperatures is below the allowable maximum of 0.5 cm. permitted by specification. In the case of sample C, the flow is 0.1 cm. higher than is permitted. It is apparent that heating to temperatures 50° F. higher than the minimum suitable pouring temperature greatly increases the flow values of all these materials.

The flow values of the materials held at the minimum suitable pouring temperatures, as shown in figure 5, are somewhat erratic, only the results on samples C and D showing a steady increase in flow with increase in time of heating.

For control of the material during the filling of expansion joints one user of large quantities of this type of filler stipulates that material represented by samples taken from the heating kettles shall not be used if the flow is greater than 2 cm. It is apparent that under prolonged heating sample C is more likely to develop unsatisfactory flow characteristics than the other materials when held at an elevated temperature for a long time.

RUBBER-ASPHALT FILLERS HAVE DUCTILITY CHARACTERISTICS DIFFERENT FROM THOSE OF USUAL JOINT MATERIALS

The softening point determination is most useful for the control of standard bituminous materials, both asphaltic and tar products. It is especially valuable in determining the resistance of these materials to the softening action of the sun or of artificial heat. It has been used, therefore, as a measure of the changes occur-

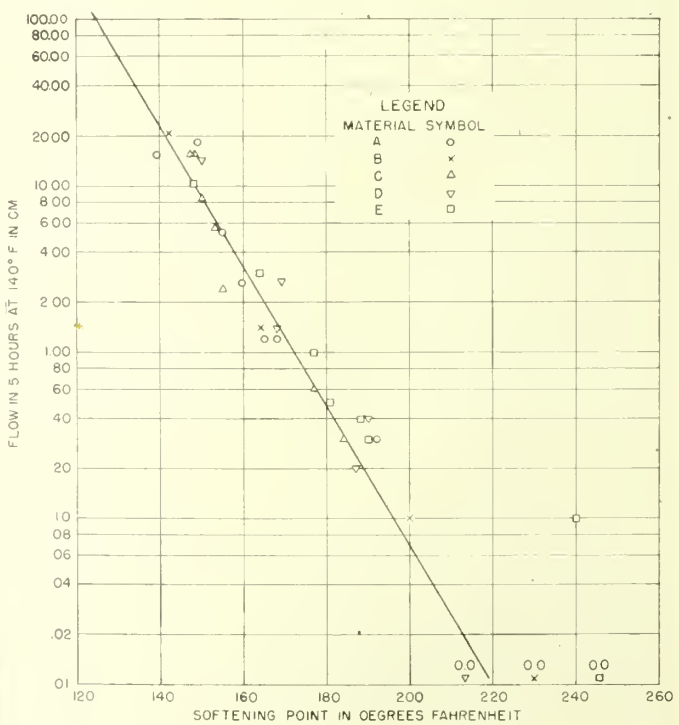


FIGURE 6.—RELATION BETWEEN SOFTENING POINT AND FLOW.

ring in these joint fillers. In making the determination of softening point of bituminous materials, a water bath is used for those softening under 176° F. and a glycerin bath for those softening over 176° F. In order to eliminate any variations in test results due to type of bath, glycerin only was used in this investigation.

The results of the softening point test plotted against the flow values obtained on the same material are shown in figure 6. A good relation between the two tests is shown. For these materials it is shown that when the softening point is over 180° F. the flow value in centimeters will be less than 0.5, and when the softening point is over 170° F. the flow will be less than 2 centimeters. Since the softening point determination can be made more easily and in less time than the flow test it would appear to be better suited as a field control test for these materials than the flow test.

The standard ductility test normally used for control of asphaltic material has been suggested by one producer as a control test for rubber-asphalt blends. It was claimed also that the contraction or rebound that occurs after the ductility specimen has broken is a measure of the resiliency of the joint filler. The ductility data given in tables 1 and 2 are shown in figures 7 and 8. In these figures the length of test specimen at time of break and the total length of the two halves of the test specimen after a 15-minute interval in the bath held at the control temperature are plotted. The percent of rebound shown in tables 1 and 2 is the difference between length of test specimen at time of break minus the total length of two halves of specimen after a 15-minute interval divided by length of test specimen at time of break multiplied by 100.

In figure 7 the effect of pouring temperature on the ductility of the various joint fillers is shown to be variable. Only in the case of sample E is there a definite trend. For this material there is a steady increase in ductility at 77° F. with increase in pouring temperature and a similar increase in ductility at 41° F. for pouring temperatures up to 500° F. The difference between ductility at 41° and 77° F. is not great for any of these materials except for the portions of sample E that were poured at 500° and 550° F. It is apparent that sample E becomes exceedingly ductile when heated to these higher temperatures. It should be noted that the producer of this material recommends that it be poured at a temperature between 380° and 400° F. which is considerably below the temperature of 500° F. which was indicated by this investigation to be suitable for its pouring. When poured at 400° F. this material is but slightly more ductile than the portions of sample C poured at 400°, 450°, and 500° F. The ductility values of the other materials do not show a consistent trend, but it is evident that the ductility values on portions poured at 500° and 550° F. are generally lower than the values of those poured at 400° and 450° F.

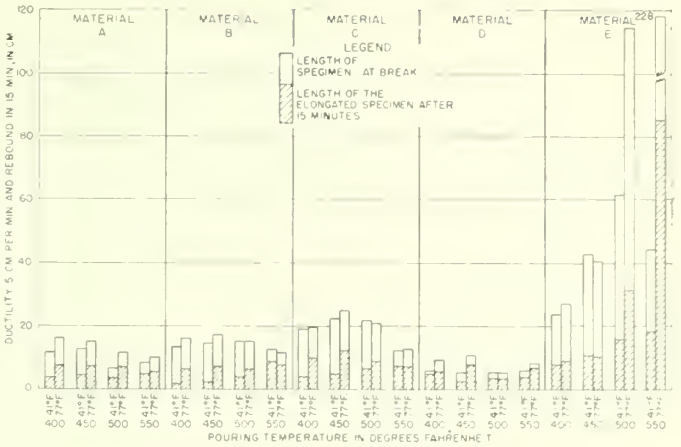

MATERIAL A
MATERIAL B
MATERIAL C
MATERIAL D
MATERIAL E
LEGEND
LENGTH OF SPECIMEN AT BREAK
LENGTH OF THE ELONGATED SPECIMEN AFTER 15 MINUTES
DUCTILITY 5 CM PER MIN AND REBOUND IN 15 MIN.,IN CM
POURING TEMPERATURE IN DEGREES FAHRENHEIT

FIGURE 7.—EFFECT OF POURING TEMPERATURE ON DUCTILITY AND REBOUND AT 41° F. AND AT 77° F.

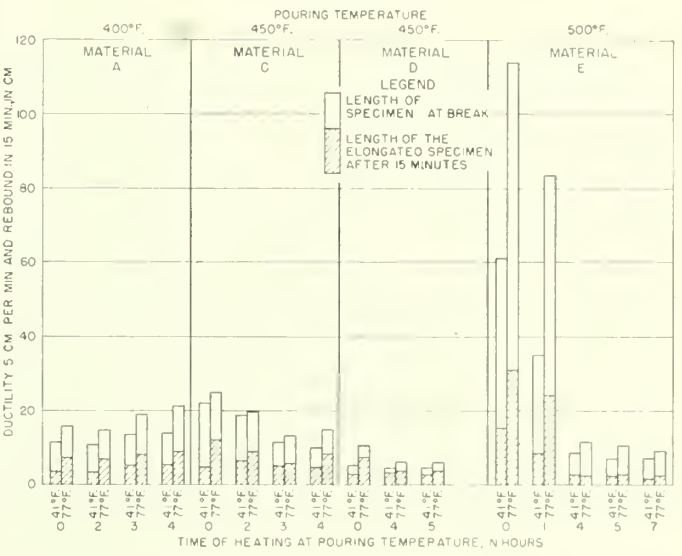


FIGURE 8.—EFFECT OF TIME OF HEATING AT MINIMUM SUITABLE POURING TEMPERATURE ON DUCTILITY AND REBOUND AT 41° F. AND AT 77° F.

RUBBER-ASPHALT FILLER RESILIENT AT LOW TEMPERATURES

The effect of time of heating on the ductility of these materials is shown in figure 8. In the case of sample A the ductility tends to increase as time of heating increases, but with the other materials the ductility tends to decrease as the time of heating increases. The latter effect is especially true of sample E. The drop in ductility after 1 hour of heating is sufficient to indicate a definite change in structure and as the heating continues the ductility values are lower than those obtained on any of the materials except sample D. Sample E, on heating to 500° F., develops ductility at 77° F. comparable to that of a steam-refined asphalt of comparable consistency. When tested at 41° F., the ductility is considerably higher than for an asphalt cement of comparable consistency. When this material is held at 500° F. for long periods, the ductility is reduced below the values obtained on the same material when heated to 400° and 450° F.

As previously indicated, the contraction in total length of the ductility specimen or rebound has been suggested as indicative of the resilient properties of these joint fillers. Table 1 shows that the percentage of rebound at 41° F. for samples A, B, and C definitely decreases as the pouring temperature increases. For samples D and E the percentage of rebound increases and then decreases as the pouring temperature increases. In 12 cases the percentage of rebound is greater for tests made at 41° F., in two cases equal to, and in six cases less than the percentage of rebound that occurs in tests made at 77° F.

Table 2 shows that with minor exceptions the percentage of rebound at 41° F. generally decreases as the time of heating increases. In 11 cases the percentage

TABLE 3.—*Ductility data on some typical blown asphalts and asphalt cements*

Identification	Penetration 100 gm., 5 sec 77° F.	Ductility test at 41° F.				Ductility test at 77° F.			
		Ductility, 5 cm./min.	Length of broken specimen after 15 min.	Rebound after 15 min.	Percent rebound	Ductility, 5 cm./min.	Length of broken specimen after 15 min.	Rebound after 15 min.	Percent rebound
Blown Asphalts:		Cm.	Cm.	Cm.	Percent	Cm.	Cm.	Cm.	Percent
1	31	0.75	0.25	0.5	67	1.5	0.25	1.25	83
2	49	.75	.4	.35	47	1.5	.75	.75	50
3	45	1.5	1.1	.4	27	2.0	1.0	1.0	50
4	35	2.0	1.4	.6	30	3.5	2.0	1.5	43
5	30	1.5	1.1	.4	27	2.0	1.25	.75	38
6	38	2.0	1.25	.75	38	2.5	1.5	1.0	40
7	42	1.5	1.1	.4	27	1.75	1.0	.75	43
9	30	2.25	1.5	.75	33	3.75	2.5	1.25	33
Asphalt Cements:									
21	41	3.25	2.3	.95	29	79	56	23	29
8	45	3.25	2.6	.65	20	103	82	21	20
13	58	5.0	4.0	1.0	20	179	106	73	41

of rebound is greater for tests made at 41° F. and in four cases less than that obtained in tests made at 77° F. In one case the percentage of rebound at each testing temperature is the same.

The test data shown in table 3 indicate that the ductility and resiliency of various blown and normally refined asphalt cements differ greatly from those of the asphalt-rubber fillers. The ductility of the blown asphalts is much lower than that of any of the rubber-asphalt materials, and those normally refined asphalt cements which have high ductility at 77° F. have a low ductility at 41° F. closely approximating that of the blown asphalts. In the case of the rubber-asphalt fillers, including those portions of sample E heated to 500° and 550° F., the ductility at 41° F. is a high percentage of the ductility at 77° F. and in some cases the low-temperature ductility equals or exceeds that at the higher temperature. Thus, the rubber-asphalt materials have a considerably greater ductility than the blown asphalts and, at low temperature, a considerably greater ductility than the normally refined asphalt cements. In contrast to the rubber-asphalt blends, table 3 shows that the percentage of rebound of the asphalts at 41° F. is always equal to or less than at 77° F. Generally the rubber-asphalt materials, as measured by the percentage of rebound, are more resilient at both 41° and 77° F. than the blown or normally refined asphalts. The better performance of these rubber-asphalt materials as joint fillers as compared to the performance of blown and normally refined asphalt cements may be attributed to the ductile and resilient characteristics which these materials possess at low temperature.

CONCLUSIONS

A comparison of the test methods required by Federal Specification SS—F—336 and of the standard test methods used for the routine examination of asphalts justify the following conclusions:

(1) Provided the average of a sufficient number of individual determinations is taken, the standard penetration test using a needle provides as satisfactory a measure of the consistency of rubber-asphalt materials as the cone penetrometer test.

(2) The softening point test can be used satisfactorily to evaluate the flow properties of rubber-asphalt materials and because of the ease and speed of making this test it is definitely more suitable as a field control test than the flow test required by the specification.

These laboratory tests show that both the temperature to which these rubber-asphalt joint fillers are heated and the time for which they are held at elevated temperatures have the following effects on their physical properties:

(1) The consistency of these materials as measured by either the cone or standard penetration at various temperatures is altered.

(2) The susceptibility to temperature change of the consistency of these materials is altered by variations in pouring temperature and time of heating.

(3) The change in susceptibility, as indicated by tests for flow and softening point, may cause the poured joint to soften and flow under summer heat.

(4) The change in susceptibility of material may cause a failure of the joint filler at low temperatures because of brittleness or lack of cohesive strength.

(5) The resilient properties may be impaired.

For these reasons it is apparent that the users of this type of material should provide for closely controlled temperature during heating with no possibility for local overheating. The optimum temperature for the particular filler should be accurately determined and maintained during the heating period within narrow limits. The batch being heated should not be held at pouring temperature for extended periods. Some effective mechanical means of pouring to expedite the placement of the filler material should be developed. Since these materials if properly handled are superior to the joint-filling and sealing materials in general use, adequate means should be taken to insure that these materials are not unduly injured before they are placed in the joint.

A revised edition of Public Control of Highway Access and Roadside Development, by David R. Levin, has recently been issued by the Public Roads Administration. The bulletin, originally issued in 1943, has been brought up to date by inclusion of pertinent State legislation through 1946. Much of the text has been rewritten and expanded. New material includes a model bill for a highway-development-rights law.

EXPERIMENTAL SAND-CLAY BASE COURSE IN NORTH CAROLINA

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by JAMES A. KELLEY, Associate Highway Engineer

THE PURPOSE of this investigation was to study, by construction and observation of an experimental road, the effect of variations in plasticity index on the service behavior of base courses composed of sand-clay mixtures. Investigations made in the laboratory by means of the circular track and the experience of various highway departments in the construction of this type of base course indicated that the plasticity index for such materials should not be greater than 6. Therefore, sand-clay mixtures having plasticity indexes both above and below 6 were specified in this investigation in order to provide information as to pavement life over a wide range of materials and to corroborate or disprove the laboratory findings.

The experimental road was constructed under contract by the North Carolina State Highway and Public Works Commission in cooperation with the Public Roads Administration. Base construction was started in August 1936 and was completed in May 1937. A bituminous surface treatment with cotton-fabric reinforcement was applied in June 1937.[1]

DESCRIPTION OF EXPERIMENTAL ROAD

The section of road selected for the experiment is located on State Highway No. 403 in Sampson and Duplin Counties, North Carolina, and is designated as Federal-aid projects 468A and B. It begins near the east city limits of Clinton and extends in an easterly direction for a distance of 14½ miles to Faison. It passes through an area of level to gently rolling topography, crossing three major stream valleys with their adjacent swampy bottom lands. The average daily traffic recorded in 1941 ranged from 660 to 1,020 vehicles.

The experimental road was divided into four sections varying from 3 to 5 miles in length. Sections having plasticity indexes of 2, 6, 8, and 14 were specified after a detailed soil survey of the roadbed and the borrow pits had been made. The sand and clay soils were combined in such a manner that the resulting mixtures had plasticity indexes within a range of ±2 of the specified value. Details of the various sections are given in table 1.

[1] The base course was originally planned in conjunction with the study of the effect of cotton-fabric reinforcement on the service behavior of thin bituminous surface treatments. The results of this cotton-fabric experiment and others are reported in *Cotton Fabric in Bituminous Construction* by Paul F. Critz; PUBLIC ROADS, vol. 24, No. 10, Oct.-Nov.-Dec. 1946, p. 251. The cotton-fabric reinforcement had no effect, beneficial or otherwise, on the performance of the road surface; and this report is devoted entirely to the influence of variations in the quality of the sand-clay base course.

TABLE 1.—*Details of experimental sections*

Section	Length	Base thickness	Plasticity index	
			Design	Range [1]
	Miles	*Inches*		
1	3.13	[2] 7	6	4–8
2	5.05	[2] 7	8	6–10
3	3.16	[2] 7	14	12–16
4	3.21	7	2	0–4

[1] Range allowed for a tolerance of ±2 from the design plasticity index.
[2] Base thickness was increased to 10 inches, owing to poor drainage conditions, on 0.38 mile of section 1, 0.23 mile of section 2, and 0.23 mile of section 3.

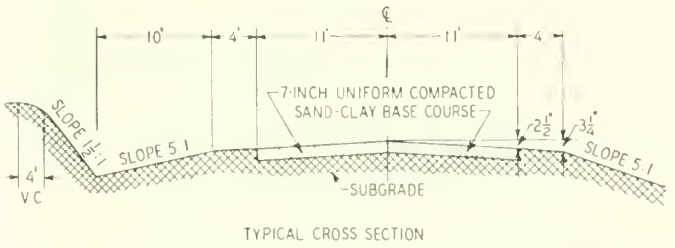


FIGURE 1.—TYPICAL CROSS SECTION OF THE EXPERIMENTAL ROAD.

A 7-inch uniform thickness of base (fig. 1) was specified for the entire experimental road. However, a soil survey disclosed the need for additional base thickness at four locations where it was evident that adverse conditions would affect the road performance very appreciably. Two of these locations were in section 2 and one each in sections 1 and 3. The base thickness was increased to 10 inches at these locations.

Sections 2, 3, and 4 had been constructed to improved line and grade in 1935 and a sand-clay surfacing of variable composition had been placed in a trench section, 18 feet wide by 6 inches deep, at that time. Preparation of the new base on these sections required the addition of sufficient material from the soil pits to increase the width to 22 feet and the thickness to 7 inches.

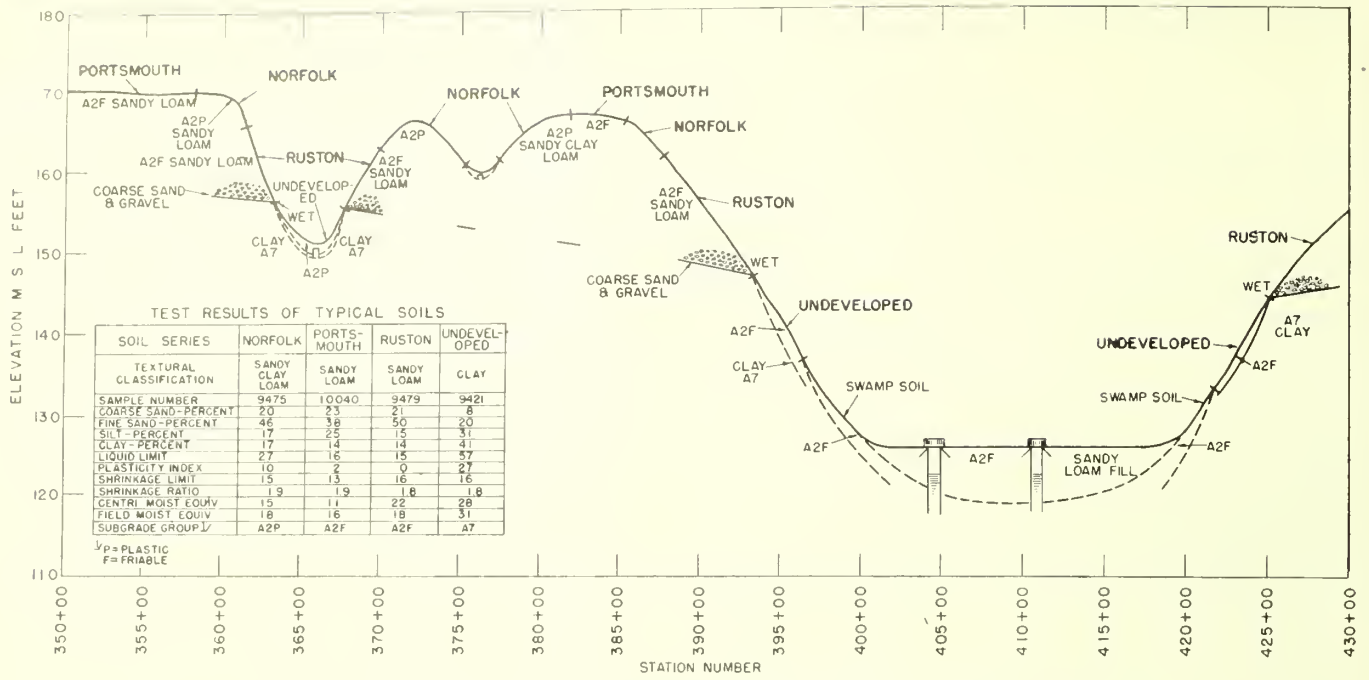
Section 1 followed generally the location of an old unsurfaced county road. Grading was necessary to provide satisfactory lines and grades prior to constructing the base course. Since there was no old sand-clay surfacing on this section, the experimental base was constructed entirely of material hauled in from selected soil pits and blended to produce the desired mixture.

DESCRIPTION OF SOILS

A detailed soil survey disclosed that the road is located almost entirely on the sandy loams of the Norfolk, Portsmouth, and Ruston series. Clays were encountered in the low swampy areas and on the slopes adjacent to the stream bottoms. The distribution of the various soils over a portion of the experimental road and the results of laboratory tests performed on typical samples are shown in figure 2. This information is representative of the soil conditions on the entire experimental road.

With the exception of the soil designated as undeveloped clay, all of the soils had physical characteristics of the A–2 friable or A–2 plastic groups. The undeveloped clay had physical properties typical of the A–7 group.

A fine sandy loam surface soil and a sandy clay subsoil are common to the profiles of the Norfolk, Portsmouth, and Ruston series. Materials for the base



TEST RESULTS OF TYPICAL SOILS

SOIL SERIES	NORFOLK	PORTSMOUTH	RUSTON	UNDEVELOPED
TEXTURAL CLASSIFICATION	SANDY CLAY LOAM	SANDY LOAM	SANDY LOAM	CLAY
SAMPLE NUMBER	9475	10040	9479	9421
COARSE SAND-PERCENT	20	23	21	8
FINE SAND-PERCENT	46	38	50	20
SILT-PERCENT	17	25	15	31
CLAY-PERCENT	17	14	14	41
LIQUID LIMIT	27	16	15	57
PLASTICITY INDEX	10	2	0	27
SHRINKAGE LIMIT	15	13	16	16
SHRINKAGE RATIO	1.9	1.9	1.8	1.8
CENTRI. MOIST. EQUIV.	15	11	22	28
FIELD MOIST. EQUIV.	18	16	18	31
SUBGRADE GROUP ↓	A2P	A2F	A2F	A7

↓P=PLASTIC F=FRIABLE

FIGURE 2.—PORTION OF SOIL TYPE PROFILE, AND TEST RESULTS OF TYPICAL SOILS.

course were obtained from borrow pits located in the soil of these series. These materials varied from non-plastic sands to sandy clays having plasticity indexes as high as 25.

FIGURE 3.—POOR PERFORMANCE OF ORIGINAL SAND-CLAY SURFACING: (*A*) OUTLET OF POROUS STRATUM CAUSING UNSTABLE AREA; (*B*) WATER FLOWING FROM SANDY STRATUM IN SIDEHILL CUT; (*C*) POORLY DRAINED AREA IN PORTSMOUTH SOIL; (*D*) UNSTABLE ROADWAY IN HIGH WATER TABLE AREA.

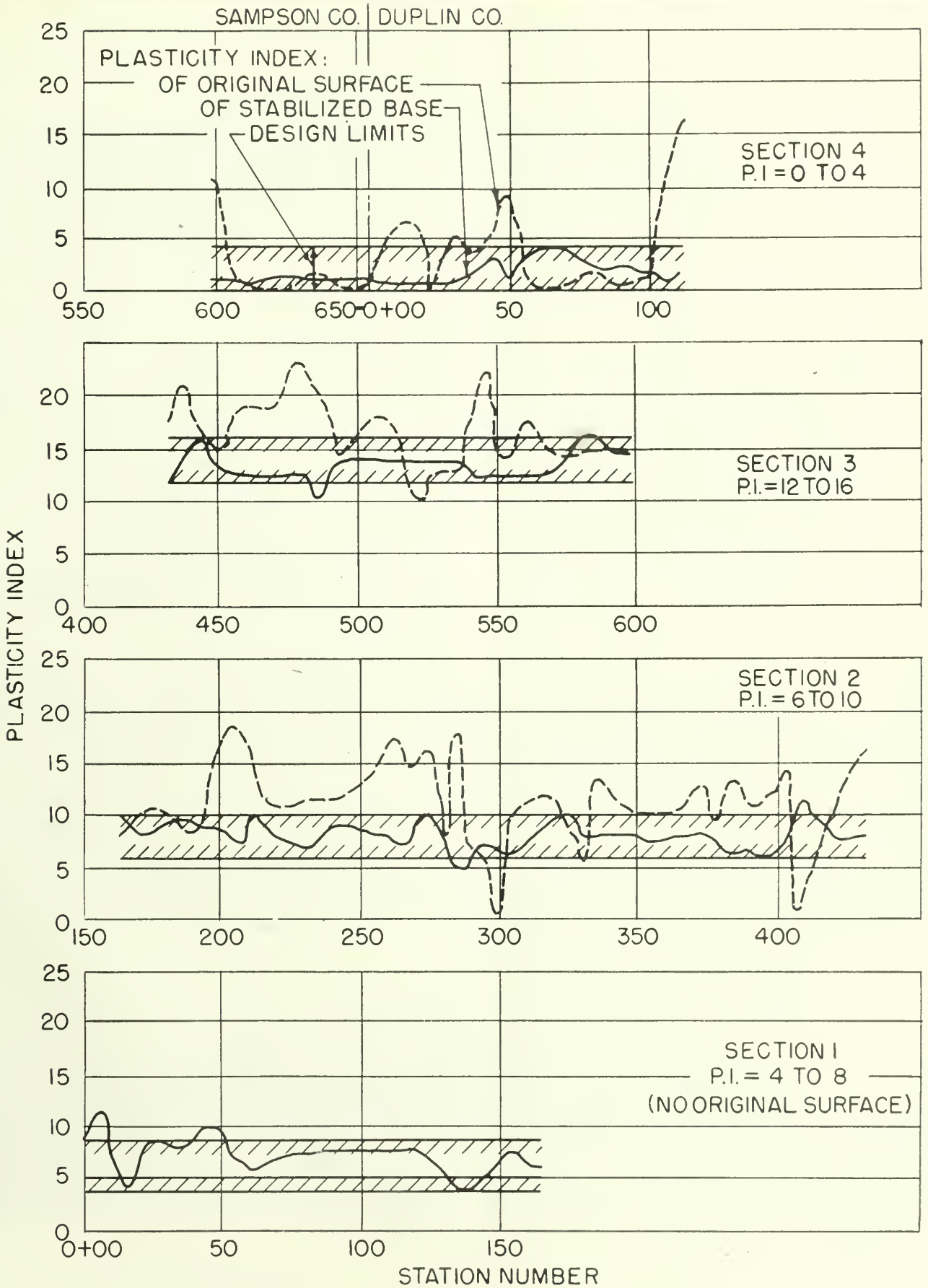


FIGURE 4.—PLASTICITY INDEX DETERMINATIONS OF THE ORIGINAL SURFACE, THE DESIGN LIMITS, AND OF THE STABILIZED BASE AS ACTUALLY CONSTRUCTED.

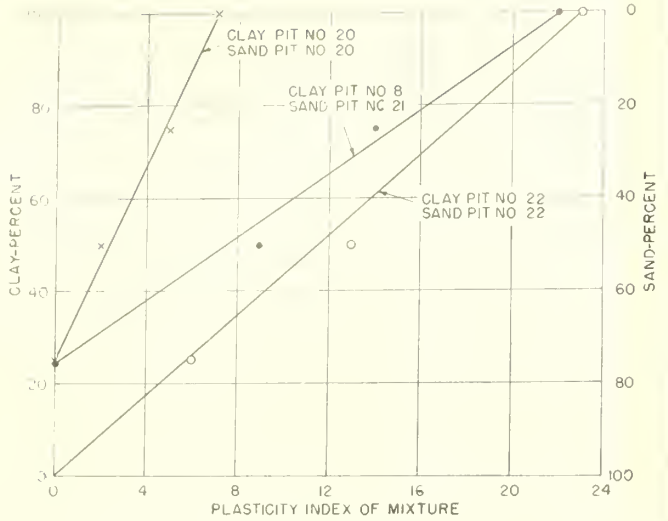


FIGURE 5.—MATERIALS PROPORTIONING CURVES BASED ON PLASTICITY INDEX.

Performance of the original sand-clay surfacing constructed in 1935 on sections 2, 3, and 4 is illustrated in figure 3. Where drainage was adequate, the road surface was maintained in satisfactory condition. However, the occurrence of springs in the roadbed, sidehill seepage, and high water table in low, poorly drained areas resulted in localized mudholes and longer sections of unstable roadway which became deeply rutted under traffic.

PROPORTIONING MATERIALS

Before calculating the amounts of sandy and clayey soil to be hauled onto the road, it was necessary to explore the borrow pits thoroughly for quantities available and to obtain samples to determine their gradations, liquid limits, and plasticity indexes. On sections 2, 3, and 4, the existing sand-clay surface was also sampled and tested and its thickness measured for determination of volume of material in place.

Plasticity indexes of the existing sand-clay surface, together with the plasticity indexes of the completed base course, are shown graphically in figure 4. Gradations and physical constants of typical base-course materials and mixtures are given in table 2.

All proportions were based on loose measurement. It was estimated that the old sand-clay surfacing would increase in volume about 50 percent when scarified and pulverized, and that the compacted thickness of the designed mixtures would amount to two-thirds of their loose thickness. To obtain a final compacted thickness of 7 inches, therefore, it was necessary to provide 10½ inches of loose material.

TABLE 2.—*Results of tests performed on samples typical of base course materials and mixtures*

Sample location	Percent passing sieve indicated				Liquid limit [1]	Plasticity index [1]
	No. 4	No. 10	No. 40	No. 200		
Borrow pit soils:						
Clay pit No. 8	100	96	56	37	52	22
Clay pit No. 20	100	100	67	32	27	7
Sand pit No. 21	100	97	56	25	NP	NP
Sand pit No. 23	100	97	62	7	NP	NP
Existing sand-clay base on old road:						
Section 2, sta. 230	100	92	54	26	37	11
Section 3, sta. 435	100	93	58	35	44	20
Section 4, sta. 650	100	86	56	23	NP	NP
Completed base course:						
Section 1, sta. 125	100	95	53	22	27	6
Section 2, sta. 220	100	98	67	23	29	8
Section 3, sta. 577	100	93	70	31	37	14
Section 4, sta. 78	100	93	73	26	18	2

[1] NP=nonplastic.

The borrow material required to produce the specified base thickness was first determined. It was then necessary to select a soil that, when combined with the material in the road, would produce a base-course mixture having a plasticity index within the range specified for the particular section. The plasticity index of the sand-clay placed on the road in 1935 was the principal factor in determining the limits of the sections and the plasticity index to be obtained in the final mixture. As shown in figure 4, most of the old material in section 2 had plasticity indexes between 9 and 15 with variations as low as 0 and as high as 18. A plasticity index of 8 was specified for this section. Similarly, section 3, with material having plasticity indexes from 10 to 22, and section 4, consisting largely of material having plasticity indexes between 0 and 5, were designed to have final plasticity indexes of 14 and 2, respectively.

A plasticity index of 6 was specified for section 1, which had no previous sand-clay surface. All of the material used in the base course on this section was hauled in from the borrow pits.

On all four sections, a tolerance of ±2 from the selected plasticity index was allowed.

Tests on a large number of samples from the borrow pits and from the old sand-clay surface of sections 2, 3, and 4 disclosed that they all had approximately the same percentage of material passing the No. 40 sieve. This simplified the problem of proportioning by eliminating the effect of differences in gradation. Under these conditions, the required plasticity indexes of the different materials are proportional to their volumes.

The relation is expressed by the equation:

$$V_F P_F = V_E P_E + V_A P_A \quad (1)$$

where: P_A = plasticity index of material to be added,
P_E = plasticity index of existing material
P_F = plasticity index of final mixture,
V_A = volume (cubic yards) of material to be added,
V_E = volume (cubic yards) of existing material,
V_F = volume (cubic yards) of final mixture.

The volumes V_A, V_E, and V_F were calculated from the designed cross section of the base and the measured cross section of the sand-clay layer existing in the old road. The plasticity index of the final mixture, P_F, was fixed by specification and the plasticity index of the material in the old road, P_E, was determined by means of laboratory tests on representative samples.

Therefore, it remained to determine the required plasticity index, P_A, of the borrow pit soil having a volume, V_A, that, when combined with the other materials, would produce the specified plasticity index in the final mixture. Transposing equation (1) and substituting $(V_F - V_A)$ for V_E gave the following equation for estimating P_A:

$$P_A = P_E + \frac{V_F}{V_A}(P_F - P_E) \quad (2)$$

FIGURE 6.—CONSTRUCTION OF SAND-CLAY BASE COURSE: (*A*) SCARIFYING OLD ROADWAY AND (*B*) REMOVING MATERIAL TO WIDEN TRENCH SECTION; (*C*) DISTRIBUTING AND (*D*) SPREADING NEW MATERIAL FROM BORROW PITS; (*E*) PLOWING ORIGINAL SURFACE AND ADDED MATERIAL; (*F*) PULVERIZING WITH DISK HARROW; (*G*) MIXING ORIGINAL AND ADDED MATERIAL WITH MOTOR GRADER; (*H*) PULVERIZING WITH DISK HARROW DURING MIXING OPERATION.

FIGURE 7. THE COMPACTED STABILIZED BASE PRIOR TO APPLICATION OF BITUMINOUS SURFACING.

When material from any one of the available borrow pits did not furnish a soil of the desired plasticity index, the materials from two pits were combined to produce a mixture having the specified plasticity index. The proportions from each of these pits were determined from curves developed from the results of tests performed on several combinations of the two materials. Figure 5 shows three of the curves used in proportioning materials for this experimental road.

In many instances, a negative value of P_A was obtained as a mathematical result in the solution of equation (2). A nonplastic material is one for which the plastic limit equals or exceeds the liquid limit. The latter condition results in a negative plasticity index. Therefore, a negative result in the solution of equation (2) shows the need for a nonplastic sandy soil as an admixture. However, there was considerable difference in the amount of reduction in plasticity index produced by different sandy soils having the same percentage passing the No. 40 sieve. This is illustrated in figure 5. The sand from pit No. 21 was more effective in reducing plasticity index than was the sand from pits Nos. 20 and 22. The plasticity index of clay from pit No. 8 was reduced from 22 to 0 by an admixture of 75 percent of sand from pit No. 21. In contrast, the plasticity index of clay from pit No. 22 was reduced from 23 to 6 with an admixture of 75 percent of sand from pit No. 22. It required an admixture of 75 percent of sand from pit No. 20 to reduce the plasticity index of clay from pit No. 20 from 7 to 0. The greater the negative value of P_A, as obtained from equation (2), the more important it was to select the sand that would produce the greatest reduction in plasticity index.

The success of the control methods used in this experiment is illustrated in figure 4. The sand-clay materials existing in the road on sections 2, 3, and 4, prior to preparation of the new base course, showed wide variations in plasticity index. Control of proportioning and mixing produced, with rare exceptions, a base course having plasticity indexes within the specified tolerance of ± 2.

CONSTRUCTION METHODS

Base construction on sections 2, 3, and 4 required the excavation of a trench 2 feet wide on each side of the existing sand-clay surfacing and the combination of the surfacing material with additional soil from selected pits to produce a base 7 inches in thickness and 22 feet wide, as shown in figure 1. This was accomplished by scarifying the shoulder material (fig. 6*A*) and removing it by means of a blade grader (fig. 6*B*), followed by scarifying and pulverizing the old surfacing to its full depth and spreading it over the 22-foot width.

Additional sand and clay soils of known characteristics as determined by test were hauled from nearby pits and deposited (fig. 6*C*) on the original surfacing material in quantities calculated to give the specified cross section after compaction. Spreading was performed by motor grader (fig. 6*D*).

Gang plows, harrows, and motor graders were used for pulverizing and mixing the different materials (figs. 6*E*, *F*, *G*, and *H*). These operations were continued until a uniform mixture was obtained and until all materials would pass through a 1¼-inch sieve and not more than 14 percent would be retained on a No. 4 sieve. The mixed materials were then spread to a uniform thickness and compacted by traffic. Motor graders maintained the surface with the proper crown during compaction. The completed base is shown in figure 7.

Construction of section 1 was performed in the same manner except that all of the material used in the base course was obtained from borrow pits.

Approximately 54 percent of the work was completed by November 1936 when the job was shut down for the winter. Section 2 and portions of sections 1 and 3 had been compacted and shaped to final cross section. In order to protect the subgrade on the remainder of the experimental road, all of the required borrow material was hauled in and partially mixed so that the road would be passable during the winter.

The road was maintained by blading and dragging during the winter. There was some loss of material due to abrasion and erosion. In some locations the loss by erosion was in such amounts that additional material was necessary to bring the base up to its required thickness.

Final mixing and compaction on these sections was started in April 1937, and the base was ready for the application of the cotton fabric and the bituminous surface in May. The total thickness of the bituminous surface was three-fourths of an inch.

BASE COURSE DENSITY

Base course densities and moisture contents were measured just prior to construction of the bituminous surface treatment. The base averaged 120.3 pounds per cubic foot at an average moisture content of 10.6 percent. This is a relative compaction of 101 percent of the density obtained in the laboratory by the standard method (A.A.S.H.O.) of test. The optimum moisture content as determined in the laboratory was 11.8 percent. Density tests made in 1945 disclosed that the relative compaction had increased to an average of 105 percent.

PERFORMANCE OF EXPERIMENTAL ROAD

Condition surveys to determine the service behavior of the base were made in December 1937, October 1938, March 1940, and September 1945. The results of these surveys are shown graphically in figure 8. A few failures were observed in December 1937, after 6 months of service. They were small local areas in sections 2 and 3 where the subgrade had become unstable as a result of seepage or high water table. By October 1938, the failures in these wet locations had developed to the point where it became necessary to excavate the subgrade, base, and surfacing and replace

FIGURE 8.—CONDITION SURVEYS AND PROFILE OF THE EXPERIMENTAL ROAD.

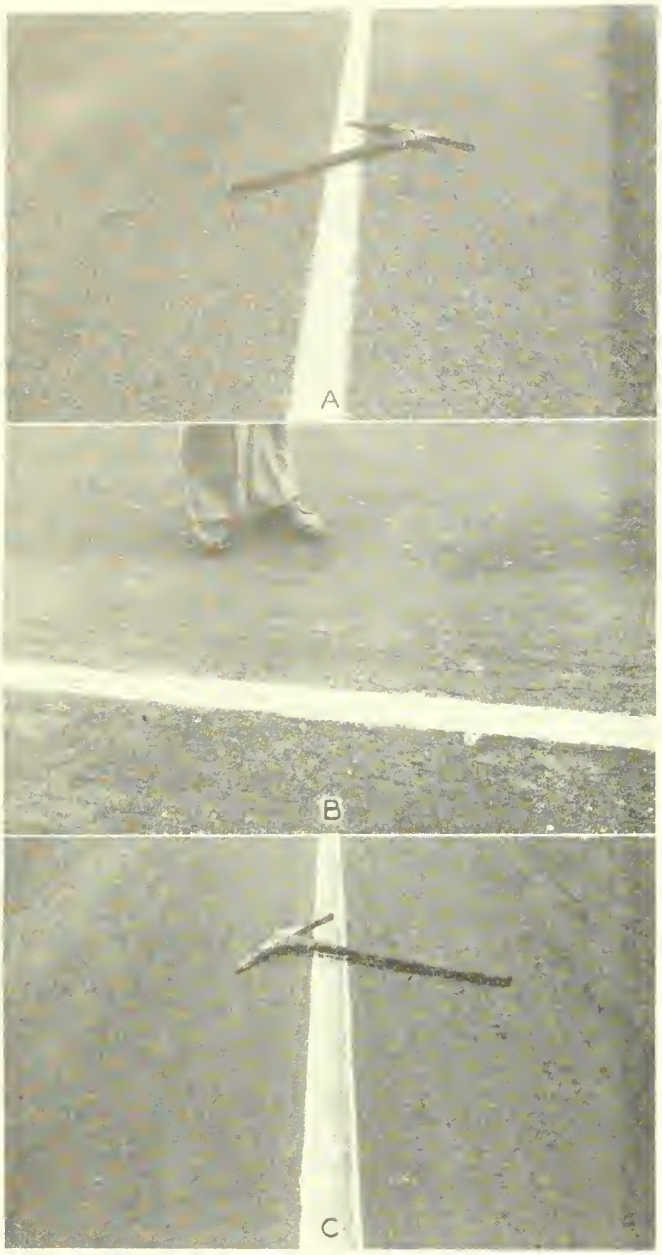


FIGURE 9.—EIGHT YEARS AFTER CONSTRUCTION, (*A*) THE ORIGINAL SURFACING WAS STILL IN GOOD CONDITION ON SECTION 4, (*B*) THE 1940 RESURFACING ON SECTION 3 HAD CRACKED, AND (*C*) THE 1944 RESURFACING ON SECTION 2 WAS IN GOOD CONDITION.

them with new material. Numerous patches were required in section 3 where the surfacing failed owing to deformations in the base course. There were no failures in section 1, and only one small patch was needed in section 4 in an area of poor drainage.

In March 1940, section 1 was still free of failures but section 4 had developed two small rough areas and required small patches. Cracking, rutting, and breakage had developed on 80 percent of section 3 where the base course had plasticity indexes ranging from 12 to 16. The rebuilt areas in section 2 were starting to crack and the breakage was spreading to the adjacent surface.

By the summer of 1940, after about 3 years of service, maintenance of the areas where failures had occurred had become such a problem that a new bituminous surface was constructed on parts of sections 2 and 4 and all of section 3. The new surface consisted of a mixture of 5½ percent of RC–2 cut-back asphalt with local sand in sufficient quantity to provide a thickness of 2 inches.

In June 1944, the new bituminous surface was extended to include most of section 2 and a small part of section 1. The new surface was applied in sections 1 and 4 because of the roughness that had developed, probably as a result of additional base compaction or fill settlement.

Failures previously described in sections 2 and 3 were showing up through the new surface when the final survey of this experimental road was made in September 1945. A small amount of cracking was also observed in sections 1 and 4.

Figure 9*A* shows the good condition of the original surface on section 4 at a point where the plasticity index of the base was 1. Figure 9*B* shows the 1940 resurfacing on a portion of section 3 where the plasticity index was 15—it will be noted that cracking had already recurred. Figure 9*C* shows the 1944 resurfacing, in good condition, at a point on section 2 where the plasticity index was 8. These photographs were made at the time of the final survey.

SUMMARY

The observations of the construction and performance of this experimental road may be summarized as follows:

1. Group A–2 plastic and A–2 friable soils comprised the subgrade on the entire length of the road. Group A–7 soils encountered in low areas were covered during grading operations with fills composed of A–2 soils.

2. Subsurface water in the form of seepage or high water table was responsible for the unstable subgrade conditions encountered. Increasing the thickness of base to 10 inches did not overcome the effects of these moisture conditions.

3. Failures occurring in the bituminous surface were the result of high plasticity index of the base course as well as unstable subgrade caused by poor drainage.

4. The base course in section 3, having a plasticity index of 12 to 16, was definitely unsatisfactory. Sections 1 and 4 with plasticity indexes ranging from 4 to 8 and 0 to 4, respectively, were satisfactory.

5. Except for failures caused by inadequate drainage, the base course on section 2, having a plasticity index range of 6 to 10, was generally in good condition. However, the fact that a new bituminous surface 2 inches in thickness was placed over practically its entire length as a maintenance measure indicates that the performance of this section was not entirely satisfactory.* In this connection, plasticity indexes greater than 6 have given good results in other instances under favorable drainage and climatic conditions. Nevertheless, the use of base-course mixtures having plasticity indexes over 6 is not advisable and it is the practice in North Carolina to require that sand-clay bases have plasticity indexes less than 6.

6. Repetition of failures in the same locations after repairs had been made demonstrates that an increase of 2 inches in the thickness of the surfacing and an increase of 3 inches in base thickness were not effective in correcting conditions resulting from inadequate drainage and excessive plasticity of the base course.

SPEED TRANSITION RAMPS ON A BRIDGED ROTARY

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by ROBERT O. SWAIN, Highway Engineer

THE EFFECT OF RAMPS to provide the speed transition between expressways and local streets was revealed in a study of a bridged rotary on the Shirley Highway just south of Washington, D. C. The Shirley Highway is a controlled-access expressway whose adequacy, like that of any expressway, is contingent on the mobility of vehicles at its interchanges. In order to obtain maximum mobility, speeds at the entrances and exits of the interchange must be consistent with speeds on the intersecting roads.

The bridged rotary on the Shirley Highway furnishes the interchange between the high-speed-designed expressway and the lower-speed streets that feed into a highly developed apartment-house community complete with a retail commercial area, as may be seen in the cover illustration. A view of the two ramps studied appears in figure 1, and their profiles are shown in figure 2.

The importance of the community adjacent to the rotary is evidenced in the distribution of traffic which is presented graphically in figure 3. More than 11,000 vehicles moved to and from the community on a peak day in June 1946. At present the highway serves local traffic only, as the completed section ends a short distance beyond the rotary. The highway, which consists of two 24-foot pavements, passes under an elliptical rotary with radii varying from 220 feet to 716 feet.

The design features [1] of the interchange are shown in figure 4, which also indicates the acceleration and deceleration ramps studied. The gradient and alinement of the ramp connections between the Shirley and the rotary were designed to enable rapid and safe acceleration and deceleration between two facilities used at different speeds. The two ramps studied were located on the north side of the interchange. The northwest ramp serves to decelerate traffic and is 615 feet long on a 3.8 percent average upgrade. The northeast ramp serves to accelerate traffic and is 510 feet in length on a 5.3 percent average downgrade.

Speed studies made during May and June 1946 disclosed that southbound traffic on the Shirley Highway averaged 37.5 mph (miles per hour) at a location just north of the bridged rotary. Upon entering the deceleration ramp, average speeds had been reduced to 35.1 mph. At the top of the ramp—the exit from the deceleration lane—the average speed was 22.5 mph, which was remarkably consistent with the average speeds of 22.8, 20.7, and 24.3 mph at three locations on the rotary.

The average speed of northbound vehicles on the rotary 300 feet from the entrance to the acceleration ramp was 24.3 mph. At the entrance to the acceleration ramp the average speed was 28.5 mph. The entrance speed to the Shirley Highway from the acceleration ramp was 36.6 mph, 1.1 mph higher than the average speed (35.5 mph) for northbound traffic on the Shirley Highway at a point just north of the interchange. The latter highway speed was observed near a railroad (service line) crossing. This may account for the slightly lower average speeds on the expressway. The average speed (28.5 mph) at the entrance to the acceleration ramp is higher than the average speed for the rotary, which may be due not only to the downgrade on this side of the rotary but possibly to the long tangent-effect approach (fig. 4) to the ramp from the southeast section of the rotary and the exceptionally good sight distance from the ramp which affords full

FIGURE 1.—THE ACCELERATION AND DECELERATION RAMPS STUDIED, SEEN FROM THE NORTH.

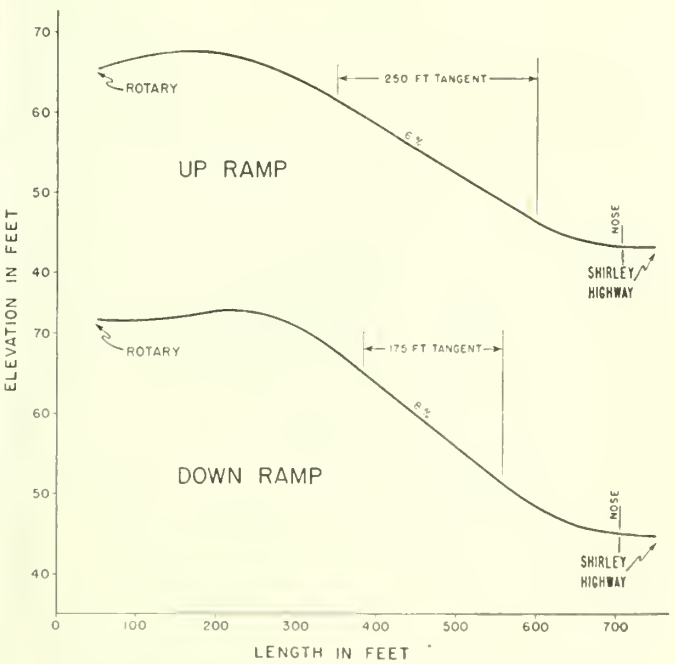


FIGURE 2.—PROFILES OF THE RAMPS STUDIED.

[1] A discussion of the design features is given in *Two-Bridge Rotary Built for Controlled-Access Interchange*, by D. W. Loutzenheiser; Roads and Streets, vol. 89, No. 4, April 1946, p. 85.

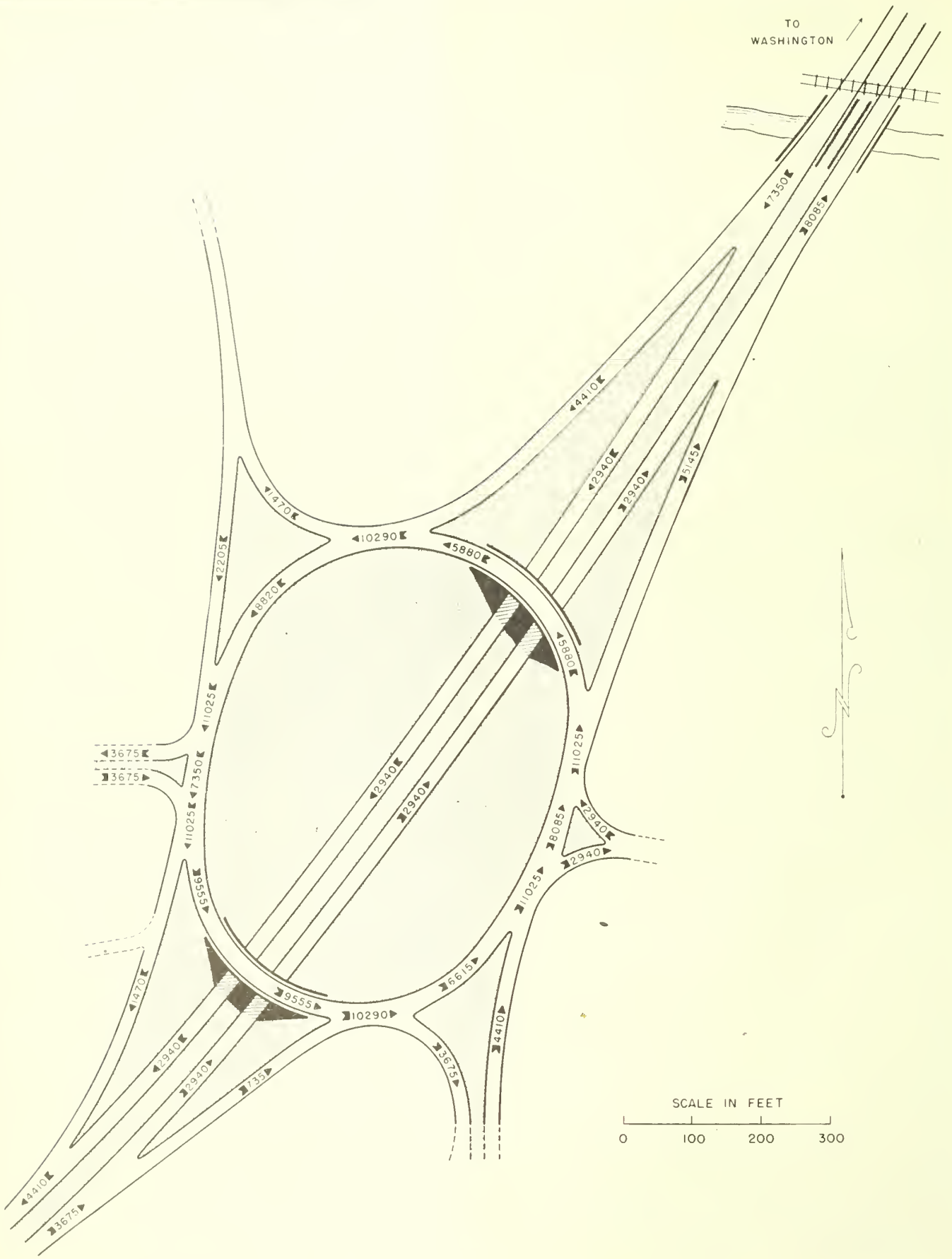


FIGURE 3.—TRAFFIC FLOW ON A PEAK DAY IN JUNE 1946 AT THE BRIDGED ROTARY, SHIRLEY HIGHWAY.

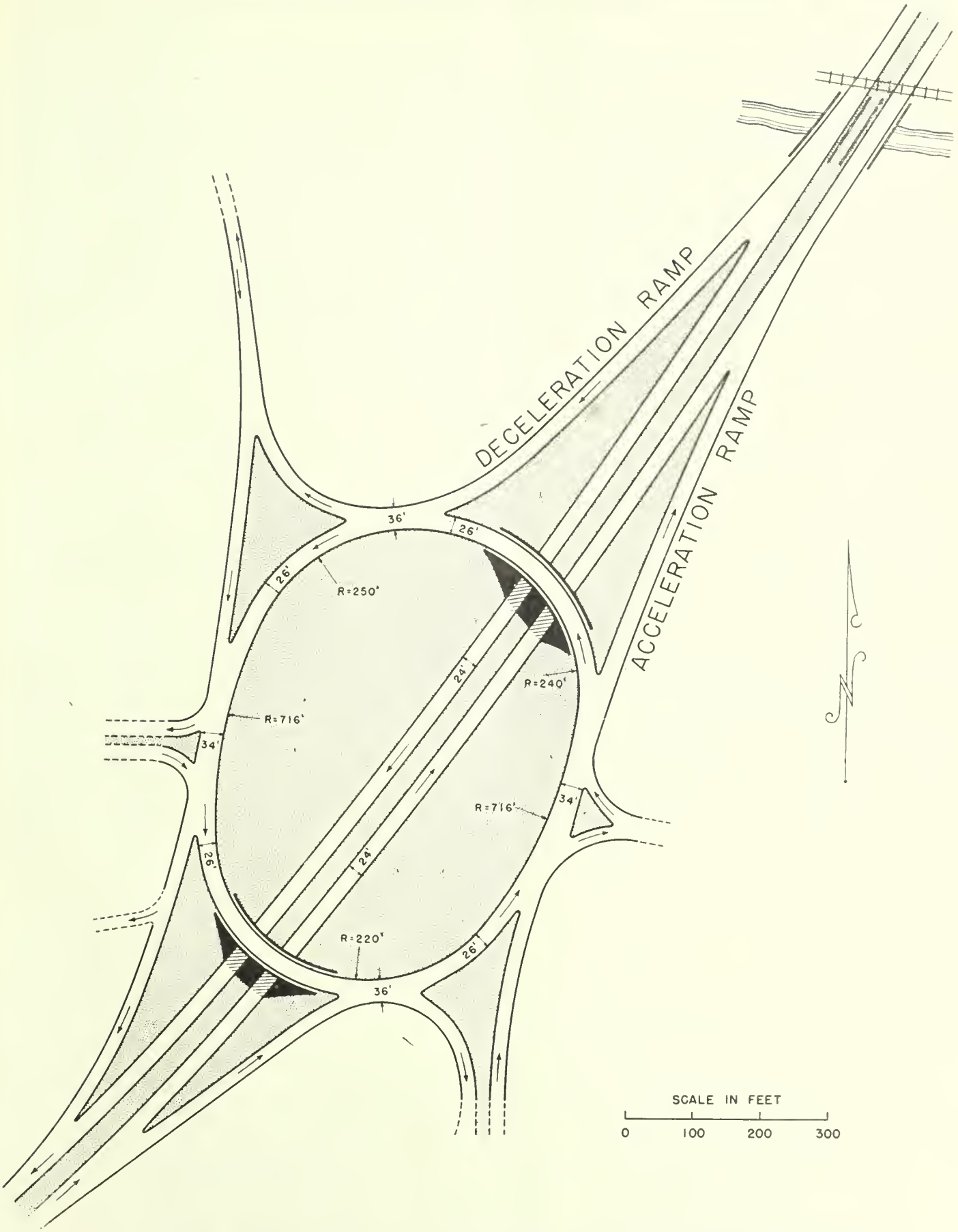


FIGURE 4.—DESIGN FEATURES AND LOCATION OF THE RAMPS STUDIED AT THE BRIDGED ROTARY, SHIRLEY HIGHWAY.

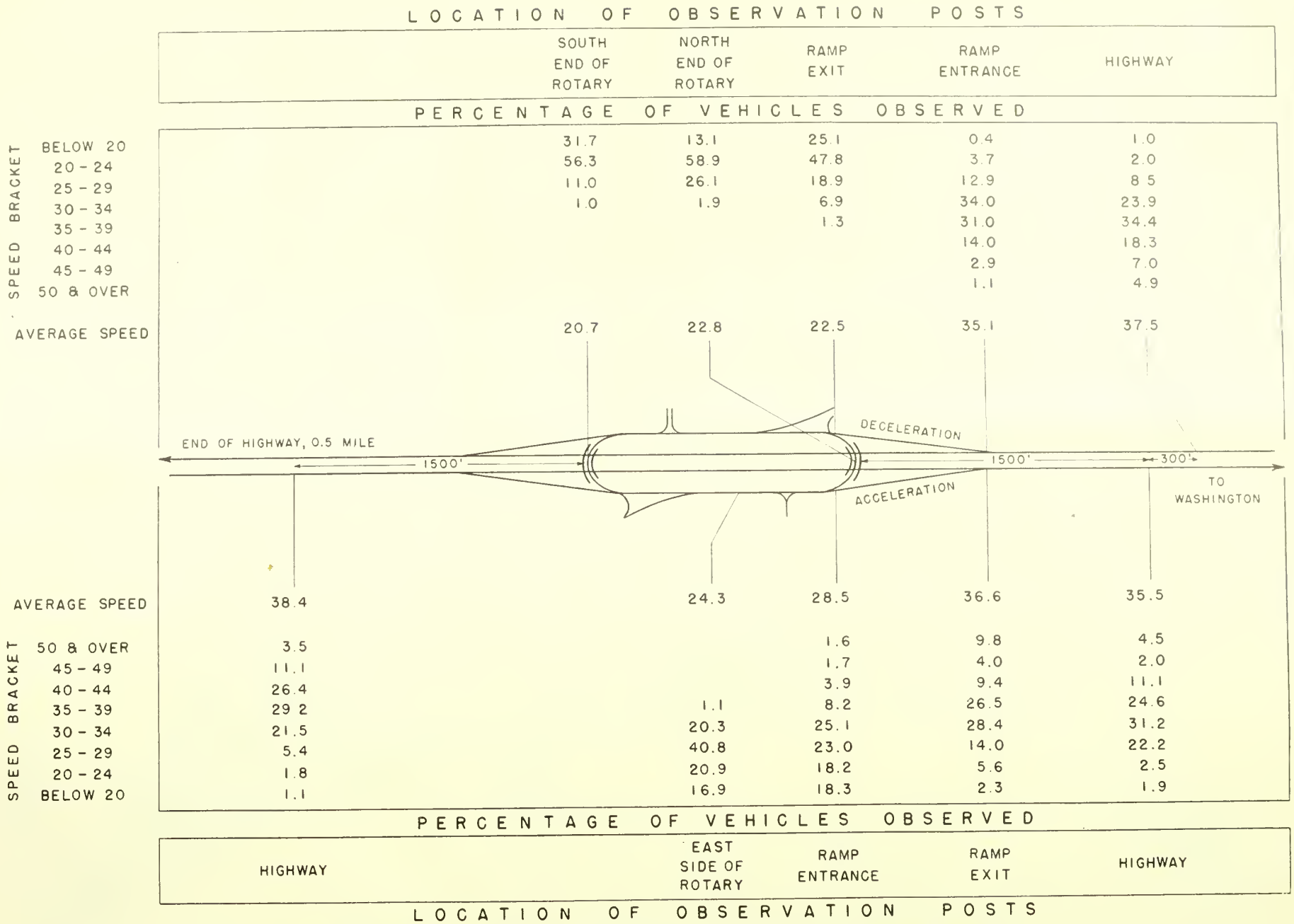


FIGURE 5.—OBSERVED SPEEDS, MAY AND JUNE 1946, AT THE BRIDGED ROTARY, SHIRLEY HIGHWAY.

view of the expressway. A summary of the observed speeds is given in figure 5.

Using the approximate average entrance speed (35 mph) at the start of the deceleration ramp, a series of test runs was made up the grade in a modern, medium-weight, six-cylinder passenger car in order to determine the effect of: (1) a closed throttle, and (2) no change in the throttle opening.

With a starting speed of 35 mph, the passenger car failed by 25 feet to reach the top of the grade when the throttle was closed by removing the foot from the accelerator. The car remained in high gear. Repeating the 35 mph entrance speed, but maintaining a constant flow of fuel (no change in the throttle opening), the passenger-car's exit speed at the end of the deceleration ramp averaged 28 mph for a large number of test runs. Entrance speeds of 50 and 60 mph, using a closed throttle on the deceleration ramp, resulted in exit speeds of 14 and 24 mph, respectively.

The passenger-car test runs were also conducted on the acceleration ramp. A point on the rotary 300 feet from the entrance to the acceleration ramp, where the average speed of 24.3 mph was observed, was used as an index and starting point. Starting at 24 mph, one series of test runs showed an average speed of 23 mph at the entrance to the ramp and 30 mph at the exit—the ramp's connection with the expressway—when no change was made in the throttle opening. Another series of test runs in which the driver was instructed to assume a "gradual and safe" acceleration resulted in an exit average speed of 51 mph after a speed of 24 mph 300 feet in advance of the acceleration ramp and a speed of 31 mph at the entrance to the ramp.

The provision of an upgrade for deceleration and a downgrade for acceleration, together with the tangent effect of the ramps, contribute to easy usage which tends to favor a depressed expressway as a suitable facility for urban use, where it is essential that vehicle speeds on the expressway be reduced materially before entering a city street and where speeds of vehicles entering the expressway must be consistent with the modal speed pattern of the expressway.

HIGHWAY STATISTICS, 1945—A NEW BULLETIN

The Public Roads Administration has recently issued a new bulletin, HIGHWAY STATISTICS, 1945, in which are presented the series of statistical and analytical tables of general interest on the subjects of motor-fuel consumption, motor-vehicle registration, State highway-user taxes, financing of State highways, and highway mileage, for the calendar year 1945. The brief text included in the bulletin highlights information of particular interest or significance in the tables.

For many years the Public Roads Administration has prepared these annual tables and has distributed them individually. This practice will be continued so that the information on each particular subject can be made available at the earliest possible date. Issuance of the collected tables in bulletin form, however, makes the data more convenient and serviceable to users of highway statistics. It is intended to publish the bulletin annually in the future.

HIGHWAY STATISTICS, 1945 is for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 35 cents a copy.

TWENTY-FIVE YEARS OF FARM TRUCK OWNERSHIP

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by C. D. BOHANNAN, Highway Economist

NEARLY 1½ MILLION TRUCKS, equal to 30 percent of all privately owned trucks registered in the United States in 1945, are owned by farmers. The number of farm trucks reported in statistical data from the 1945 census of agriculture is 1,490,300, representing a 42.3 percent gain over 1940. This gain takes

TABLE 1.—*Number of trucks on farms, and number and percent of farms reporting trucks—1920 to 1945* [1]

Division and State	Number of trucks on farms					Farms reporting trucks							
						Number of farms				Percent of all farms			
	1945		1940	1930	1920	1945	1940	1930	1920	1945	1940	1930	1920
	Number	Percent of all trucks [2]											
United States	1,490,300	30.8	1,047,084	900,385	139,169	1,299,350	944,184	845,335	131,551	22.2	15.5	13.4	2.0
Geographic divisions:													
New England	74,151	25.2	51,207	37,972	8,119	59,372	42,670	33,328	7,284	39.5	31.6	26.7	4.7
Middle Atlantic	148,632	20.3	116,714	120,789	22,011	121,457	101,093	109,756	20,472	35.0	29.0	30.7	4.8
East North Central	240,161	28.2	191,394	198,172	26,074	220,357	177,085	189,040	24,942	23.1	17.6	19.6	2.3
West North Central	285,597	43.5	199,234	180,857	33,375	261,765	186,429	174,764	32,331	25.4	17.1	15.7	2.9
South Atlantic	175,872	29.3	116,297	94,946	15,787	153,630	101,295	88,019	14,716	14.7	10.2	8.3	1.3
East South Central	105,736	34.2	71,830	45,568	5,153	97,566	66,985	43,234	4,869	10.2	6.5	4.1	0.5
West South Central	187,775	32.7	121,788	96,791	9,455	168,660	112,180	92,300	8,960	19.2	11.6	8.4	0.9
Mountain	118,528	43.3	75,552	55,742	7,589	98,600	67,083	52,488	7,168	46.4	28.7	21.8	2.9
Pacific	153,848	28.2	103,068	69,548	11,606	117,943	86,364	62,406	10,809	41.8	31.3	23.8	4.6
New England:													
Maine	17,849	35.4	13,118	10,781	1,120	15,229	11,698	9,879	1,061	36.1	30.0	25.3	2.2
New Hampshire	9,191	29.1	6,469	4,539	717	7,486	5,489	4,051	663	39.8	33.2	27.2	3.2
Vermont	9,567	88.0	6,172	5,035	616	8,386	5,451	4,625	576	31.7	23.1	18.6	2.0
Massachusetts	18,751	17.1	12,465	9,572	3,535	14,589	9,934	7,982	3,136	39.4	31.1	31.2	9.8
Rhode Island	3,142	14.5	1,982	1,701	536	2,115	1,453	1,366	471	58.7	48.2	41.1	11.5
Connecticut	15,651	22.6	11,001	6,344	1,595	11,567	8,645	5,425	1,377	52.0	40.8	31.5	6.1
Middle Atlantic:													
New York	69,141	22.7	55,285	58,974	9,259	57,049	48,522	54,084	8,636	38.2	31.7	33.8	4.5
New Jersey	23,220	15.7	17,106	14,753	3,380	16,338	13,590	12,504	3,075	62.3	52.6	49.3	10.4
Pennsylvania	56,271	20.2	44,323	47,062	9,372	48,070	38,981	43,168	8,761	28.0	23.1	25.0	4.3
East North Central:													
Ohio	42,808	21.4	35,169	39,210	7,319	38,670	31,872	36,693	6,960	17.5	13.6	16.7	2.7
Indiana	38,411	26.1	29,732	30,037	3,671	35,260	27,409	28,468	3,501	20.0	14.9	15.7	1.7
Illinois	56,629	26.6	42,515	40,371	6,154	52,600	39,970	38,877	5,907	25.8	18.7	18.1	2.5
Michigan	41,303	27.8	33,095	36,768	4,886	37,304	30,311	34,894	4,681	21.3	16.2	20.6	2.4
Wisconsin	61,010	42.7	50,883	51,786	4,044	56,523	47,523	50,108	3,893	31.8	25.4	27.6	2.1
West North Central:													
Minnesota	47,413	41.2	38,617	36,557	3,803	44,170	36,101	35,503	3,677	23.4	18.3	19.2	2.1
Iowa	37,386	36.5	26,352	32,669	8,910	35,482	24,947	31,874	8,669	17.0	11.7	14.8	4.1
Missouri	47,225	30.8	31,771	20,132	5,059	44,241	29,845	19,215	4,878	18.2	11.7	7.5	1.9
North Dakota	36,674	74.4	21,518	16,990	774	32,494	19,937	16,502	743	46.7	27.0	21.2	1.0
South Dakota	22,230	59.8	14,269	14,816	4,353	20,487	13,472	14,485	4,249	29.8	18.6	17.4	5.7
Nebraska	33,761	45.4	24,090	26,045	6,548	30,792	22,504	25,176	6,333	27.6	18.6	19.4	5.1
Kansas	60,908	48.8	42,617	33,648	3,928	54,099	39,623	32,009	3,782	38.3	25.3	19.3	2.3
South Atlantic:													
Delaware	3,800	28.8	2,567	2,996	304	3,110	2,215	2,781	283	33.4	21.6	28.6	2.8
Maryland	19,197	27.1	13,058	11,284	2,805	16,080	11,451	10,205	2,556	38.9	27.2	23.6	5.3
District of Columbia	35	0.3	39	81	29	24	32	57	29	60.0	49.2	54.8	14.2
Virginia	31,956	36.0	23,272	19,459	2,544	27,966	20,818	17,964	2,389	16.2	11.9	10.5	1.3
West Virginia	16,285	26.7	12,445	7,432	936	14,843	11,485	6,936	886	15.2	11.6	8.4	1.0
North Carolina	32,924	32.8	20,621	18,558	2,671	30,489	19,279	17,613	2,551	10.6	6.9	6.3	0.9
South Carolina	15,348	27.7	8,242	6,966	1,736	13,296	7,392	6,575	1,609	9.0	5.4	4.2	0.8
Georgia	34,688	34.2	21,693	15,967	3,145	31,250	19,952	15,036	2,913	13.8	9.2	5.9	0.9
Florida	21,639	22.7	14,360	12,203	1,617	16,572	11,668	10,852	1,500	27.1	18.7	18.4	2.8
East South Central:													
Kentucky	27,194	33.5	19,100	7,188	1,538	25,175	17,830	6,740	1,455	10.6	7.1	2.7	0.5
Tennessee	26,326	32.8	18,968	9,039	1,430	24,659	17,727	8,563	1,362	10.5	7.2	3.5	0.5
Alabama	23,949	31.3	15,257	12,838	1,180	21,939	14,094	12,169	1,114	9.8	6.1	4.7	0.4
Mississippi	28,267	39.5	18,565	16,503	1,005	25,793	17,334	15,762	938	9.8	6.0	5.0	0.3
West South Central:													
Arkansas	33,134	42.2	19,674	11,000	1,027	30,304	18,441	10,526	973	15.2	8.5	4.3	0.4
Louisiana	20,974	26.8	17,005	9,281	874	18,637	15,663	8,769	793	14.4	10.4	5.4	0.6
Oklahoma	44,381	40.4	28,402	23,930	2,155	40,910	26,660	23,039	2,070	24.8	14.8	11.3	1.1
Texas	89,286	29.0	56,707	52,580	5,399	78,809	51,416	49,966	5,124	20.5	12.3	10.1	1.2
Mountain:													
Montana	29,388	61.6	21,037	14,615	1,225	23,661	18,332	13,783	1,167	62.7	43.8	29.0	2.0
Idaho	19,096	49.6	12,002	6,281	837	16,576	10,941	5,986	779	40.0	25.1	14.4	1.9
Wyoming	8,917	44.1	6,341	4,108	591	7,424	5,553	3,820	554	56.8	37.0	23.9	3.5
Colorado	28,794	40.1	16,850	16,918	3,016	24,057	15,119	16,052	2,884	50.5	29.4	26.8	4.8
New Mexico	11,902	40.0	7,117	5,328	593	10,256	6,368	5,024	552	34.5	18.7	16.0	1.8
Arizona	6,859	23.3	4,284	3,062	581	5,008	3,561	2,719	527	38.1	19.3	19.2	5.3
Utah	10,969	40.9	6,238	4,189	572	9,720	5,801	3,986	544	36.9	22.8	14.7	2.1
Nevada	2,603	27.4	1,683	1,241	174	1,898	1,408	1,118	161	55.4	39.4	32.5	5.1
Pacific:													
Washington	40,031	39.1	28,228	18,836	3,371	32,730	24,886	17,622	3,172	41.0	30.5	24.9	4.8
Oregon	28,118	34.1	16,825	9,741	1,819	23,253	14,904	9,110	1,728	36.8	24.1	16.5	3.4
California	85,696	23.8	58,015	40,971	6,416	61,960	46,574	35,674	5,909	44.6	35.1	26.3	5.0

[1] Data from censuses of agriculture for 1920, 1930, 1940, and 1945.
[2] "All trucks" are truck registrations by States as reported to Public Roads Administration, excluding publicly owned vehicles (table MV–1, 1945).

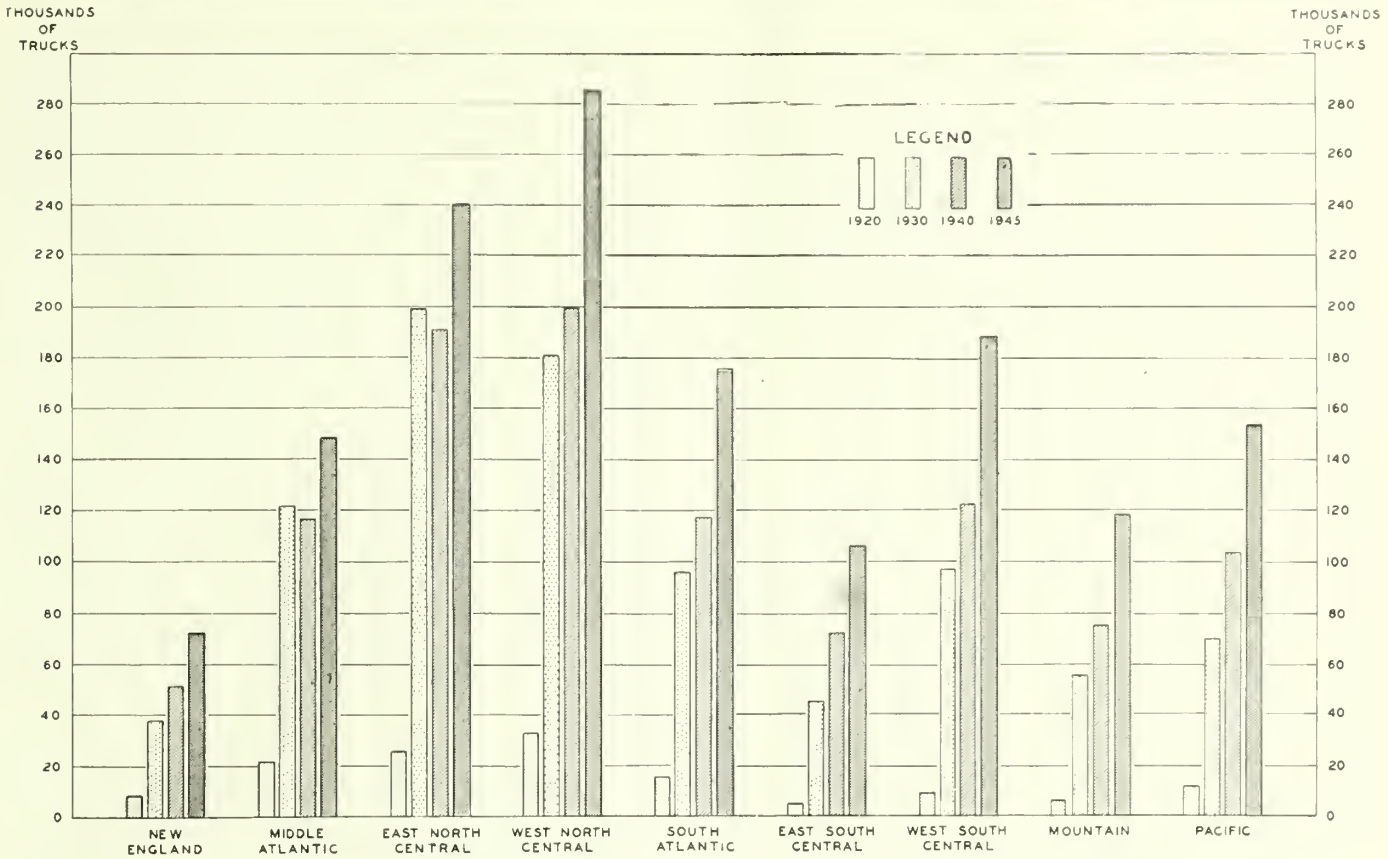


FIGURE 1.—NUMBER OF TRUCKS ON FARMS, BY GEOGRAPHIC DIVISION—1920 TO 1945.

on added significance when it is recalled that as a war measure civilian truck manufacture and sales were frozen after March 1942.

Another indication of the growth of farm truck ownership is the increase in the percentage of farms reporting trucks owned, from 15.5 percent in 1940 to 22.2 percent in 1945.

Twenty-five years ago (1920) only 2 percent of the Nation's farms reported trucks. In that year the total number of farm trucks was 139,169 or 13.8 percent of the

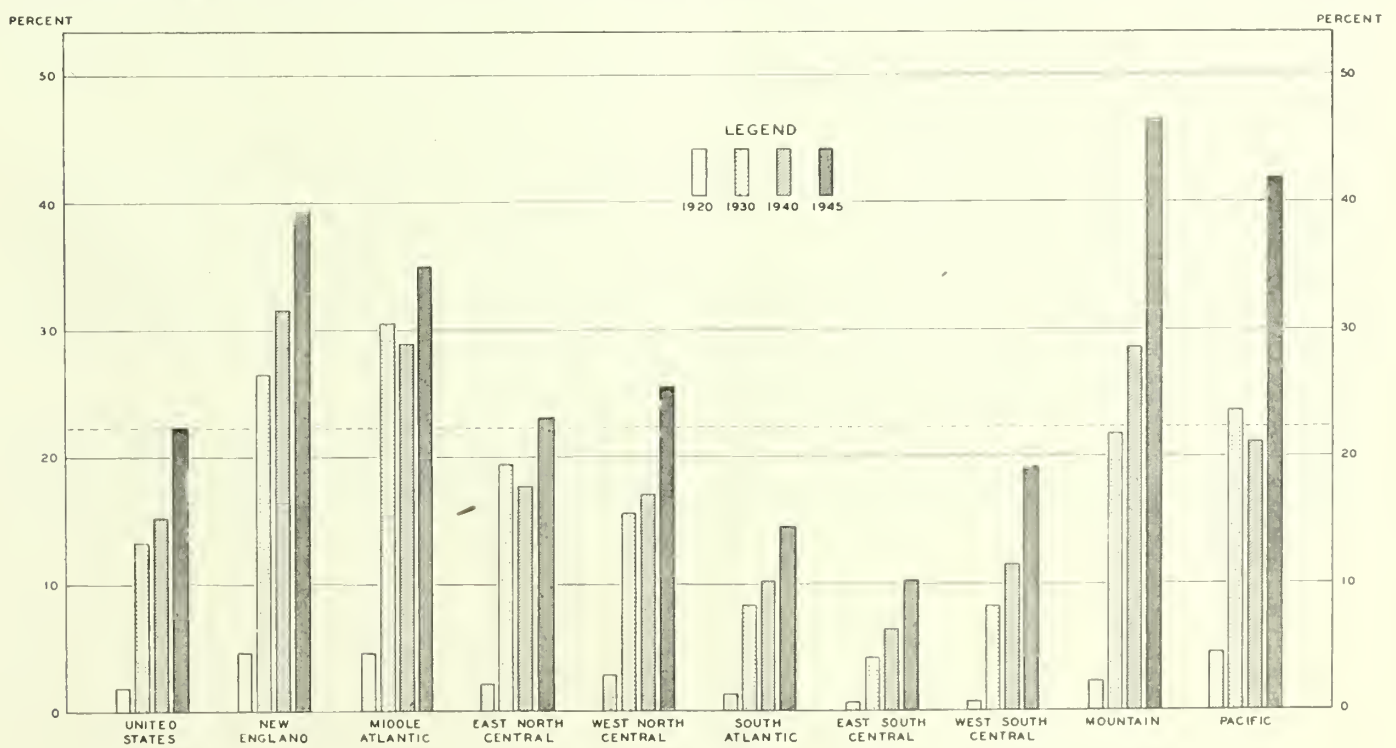


FIGURE 2.—PERCENT OF FARMS REPORTING TRUCKS OWNED, BY GEOGRAPHIC DIVISION—1920 TO 1945.

estimated total truck registration in the United States.

Table 1 reports detailed data, by State and geographic division, on the number of trucks on farms and on the number of farms reporting trucks owned, from the censuses of agriculture for 1920, 1930, 1940, and 1945. Figures 1 and 2 show graphically the 25-year change in the number of trucks on farms and the percentage of all farms reporting trucks, for each geographic division.

Significant variations from one geographic division to another and one State to another are readily apparent. The State data also permit some interesting comparisons with the national averages. For example, the 1945 data show that in 15 States the number of farm trucks equals 40 percent or more of all trucks registered, as compared to the national average of 30 percent, ranging as high as 88 percent in Vermont, 74 percent in North Dakota, and 62 percent in Montana.

Texas, containing more farms than any other State just as it exceeds all others in total area, also leads in the number of farms reporting trucks, 78,809, and in the number of trucks on farms, 89,286. California, although only 60 percent the size of Texas and with less than half as many farms, reported the next highest number of trucks on farms, 85,696. In Texas, furthermore, only 20 percent of all farms reported having trucks, while in California 45 percent had trucks.

Twenty-one States showed an increase of over 50 percent in the number of farm-owned trucks from 1940 to 1945. The largest percentage gain was 86 percent in South Carolina, representing, however, an increase from only 8,242 trucks in 1940 to 15,348 in 1945. The 20-percent increase in Wisconsin represented a gain from 50,883 trucks in 1940 to 61,010 in 1945.

In only seven States did half or more of the farms report having trucks. One of these was New Jersey, which in 1940 was the only State in which over 50 percent of the farms had trucks.

On the whole, this record of 25 years of farm truck ownership portrays the growing importance of trucks in farm operations. Adequate Nation-wide data are not available to show in detail the nature of the uses to which these farm trucks are put, the size and kind of trucks, or the average annual miles of travel. However, sample studies and certain related data do throw some light on these and kindred subjects.

In most sections of the country the farmers' preference is for a 1½-ton truck—that is, a truck large enough to haul a real load and yet not so large as to represent unneeded capacity and expense.

A WPA technology report [1] on farm machinery and equipment indicated that 80 percent of farm-owned trucks were in the 1- to 2-ton capacity group. Further, a detailed analysis of State-by-State estimated needs for additional farm trucks during the war period showed a very heavy preponderance of preference for the 1½-ton size.

A considerable part of the use of farm trucks is on the farm, as distinct from on the highway. Nation-wide data on this interesting subject are lacking, and apparently very few sample studies have been made.

One study made by the Agricultural Experiment Station at Purdue University [2] in a group of central Indiana counties showed that in 1937, farmers who owned trucks used them on the average of 197 days during the year. On-the-farm use accounted for 55 percent of the time the trucks were used and for 28 percent of the average 4,061 miles traveled.

Although much of the use of farm trucks is in hauling products to shipping points and markets and the return haul of farm and household supplies, many farmers in some of the heaviest producing agricultural counties do not own trucks.

Studies of the marketing of farm products have shown that for some products the haul from farm to market is principally by for-hire carriers or by buyers who pick up at the farm. In a study of the marketing of livestock in the Corn Belt States [3] it was found that of all hogs marketed by farmers in 1940 only 21 percent were hauled in farm-owned trucks, 66 percent were hauled by for-hire truckers, and 13 percent by buyers. A Wisconsin study covering the year 1937 showed that only about 30 percent of the farmers hauled their own milk.

In some areas a large share of the feeds, fertilizers, and petroleum products consumed on farms is delivered to the farms either by the dealers or by for-hire truckers.

This movement of commodities to and from the farms in other than farm-owned trucks emphasizes the fact that the condition of farm roads is of vital concern not only to the farmers who live along them but also to those business firms and individuals who buy from and sell to farmers, and to the for-hire truckers who furnish the necessary transportation services.

The exact extent of future farm truck ownership is, of course, impossible to predict. It is conceivable that changes in kinds of farm production and increased availability of for-hire truck service, especially in some areas, may operate to lessen the desirability of farm truck ownership.

On the other hand, some changes in farm production enterprises and increased rural industrialization may call forth an increased demand for trucks on farms.

It seems safe to assume that, given a fair degree of farm prosperity and continued improvement in farm and other local rural roads, there will be a continued steady demand for trucks by farmers. This demand is likely to be intensified if the manufacturers are able to develop trucks especially adapted to use on the farm as well as on the road.

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[1] *Changes in Farm Power and Equipment: Tractors, Trucks, and Automobiles*, by E. G. McKibben and R. A. Griffin; Works Progress Administration, National Research Project Report No. A-9, December 1938, p. 44.

[2] *The Use of Farm Trucks in Marketing Farm Products in Central Indiana*, by T. K. Cowden; Purdue University Agricultural Experiment Station Bulletin No. 443, August 1939, pp. 11 and 12.

[3] *Marketing Livestock in the Corn Belt:* South Dakota State College Agricultural Experiment Station Bulletin No. 365, November 1942, p. 166.

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